STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF CONSTRUCTION
OFFICE OF TRANSPORTATION LABORATORY

WAVE EQUATION ANALYSIS OF PILES INSTALLED WITH DIESEL HAMMERS

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NOTICE

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This Interim Report is prepared in fulfillment of the federally funded research project titled "Wave Equation Analysis of Piles Installed with Diesel Hammers," Project No. F82TL08, Expenditure Authorization No. 54322-632463.

Background

The original objective of this project, as stated in the research proposal, was to develop a relationship between soil type and values of damping coefficient - an important input parameter for the WEAP (Wave Equation Analysis of Pile Driving) computer program. Implied by the proposal were the needs to evaluate the WEAP program for construction control of pile driving, to make the WEAP program operational on Caltrans computers and to train personnel in its use.

In the process of pursuing this project, Caltrans became familiar with and installed the WEAP program on its mainframe computer. The project provided the circumstances under which Caltrans personnel became familiar with the operation and nuances of wave mechanics as applied to pile driving.

In analyzing the data, the researchers developed confidence in the WEAP procedure, and recognized that this method represents an improvement over the ENR formula for predicting pile capacity. The work also indicated changes which must be made in contract documents and administration in order to implement WEAP into the Caltrans construction program.

As an offshoot of the work performed with the WEAP program, Caltrans undertook (with separate funding) the purchase of a Pile Driving Analyzer (PDA). This device provides an in-the-field means of computing pile capacity using data measured during pile driving and theory similar to that used by the WEAP program. A nucleus of TransLab Geotechnical Branch personnel have been trained to use the analyzer. The PDA was used at many of the research sites and is now a routine part of construction control.

Also, as a result of this work, Caltrans static pile load test procedures are being modified, and consideration is being given to establishing a rational approach to defining an ultimate pile capacity which considers the properties of the pile being tested. Work is being undertaken to facilitate the testing of piles to the ultimate capacity of the foundation soils.

Unfortunately, incomplete familiarity with the WEAP program at the inception of this project precluded the formulation of a comprehensive work plan. This led to difficulties during the analysis phase when it was realized that the data lacked sufficient detail to support the development of damping coefficient relationships. As a result, the validity of the reported damping coefficient relationships are questionable.

The work was carried out under the constraints of standard construction practices and using existing Caltrans pile load test procedures. It was believed that by using standard pile load test procedures, time and money could be saved while still generating satisfactory data. However, this proved to be a false hope since the amount of soil set-up that occurs during the lapse of time (up to one week) between pile driving and pile load testing is unknown, and because many of the pile load tests were not carried to a sufficiently high displacement. Also, the lack of comprehensive pile hammer system data for many of the research sites made it difficult to eliminate significant variables. These circumstances combined to make the reported conclusions of limited value.

Work performed for this project used the 1980 version of the WEAP program, and the reported results do not apply to the newer version of WEAP, titled WEAP86, released in 1986. WEAP86 incorporates substantial improvements to the hammer and dynamic soil resistance models in order to take advantage of innovations in these areas. It also offers the significant enhancement of accounting for the effects of residual stresses in the pile after driving. All future work should use WEAP86.

The draft final report is attached for general information purposes only. The reported relationships should not be used for design or construction control purposes.

Expenditures

Total expenditures for this project were $43,249. Expenditures were only for personnel costs. Travel was charged against construction projects and no equipment was purchased.

Expenditures broken down by fiscal year are as follows:

F.Y.	Salary	Time
1982-83	$ 6,009	252 hr
1983-84	6,891	297
1984-85	24,182	918
1985-86	6,167	284
	$43,249	1751 hr

Recommendations for Future Work

In order to fully achieve the original objectives of this project, Caltrans will have to address not only the shortcomings previously outlined, but should deal with the contractual and organizational difficulties caused by the increased informational demands associated with using wave equation analysis to control pile driving. A logical method of approaching this would be to break the project down into several steps and provide interim reports on each step.

One step would be to make construction evaluations of the contractual difficulties arising from the use of the WEAP86 program. These evaluations would be made by structures construction personnel on several test projects with

the overall objective of testing different methods of WEAP86 program implementation. TransLab Geotechnical personnel would provide technical assistance.

Another step would be to perform comprehensive pile load testing and compare the results to WEAP86 predicted ultimate pile capacities and ENR predicted safe pile capacities. It is conceivable that this phase would extend several years so that a statistically relevant data base can be assembled. Interim reports would be published every two years.

At the conclusion of these studies, a cost benefit relationship can be established. If the results indicate wave equation analysis to be an improvement in construction control of pile driving, then WEAP86 can be fully implemented. At that time, additional efforts to develop a relationship between soil damping and a measurable soil property may be appropriate.

A fully developed research proposal for further evaluation of the WEAP86 program will be made in the near future. Any work undertaken will, of course, be contingent on Division of Structures participation.

ATTACHMENT

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A parametric study of the WEAP program was made with the determined J_c values for the ten sites. The results show that reasonable answers can be obtained by the use of straightline fit correlation between J_c and SPT values.

17. KEY WORDS
Pile bearing capacity, wave equation procedure, Weap Program, Pile Dynamic Analyzer, Case-Goble Method, soil damping constant, standard penetration test, static load test, force, velocity.

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TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

TABLE OF CONTENTS (Continued)

LIST OF TABLES

LIST OF FIGURES

APPENDIX

APPENDIX (Continued)

CHAPTER 1

INTRODUCTION

A great deal of research on the wave equation procedure for pile driving has been conducted in recent years to establish a rational method of analysis that is consistent with engineering theory and observed behavior. The wave equation procedure describes the movement of stress waves in a pile when the pile top is struck by a heavy hammer (the stress waves measured during actual pile driving are force and velocity waves). The measured characteristics of the generated stress waves can be used to calculate static bearing capacity if a quantity called the soil damping constant is known. The Case Method soil damping constant (J_c) is a convenient way to account for the different dynamic soil resistances of the different soil types encountered during pile driving. The wave equation procedure can also be used to evaluate hammer performance.

Traditional energy formulas (such as the Engineering News Formula) do not consider the dynamic resistance of the soil(1). Therefore, it is advantageous to use the wave equation technique in order to account for these forces. However, despite considerable use of the wave equation method, there is still a need to provide additional insight into the damping behavior of soils associated with pile driving.

Various computer programs have been developed based on the wave equation technique for pile driving. Two of these programs will be discussed in this paper; the Wave Equation Analysis of Piles (WEAP)(2) and the Pile Driving Analyzer (PDA)(3).

The WEAP program models the piling system (i.e., hammer, cap, pile, and soil) using information supplied by the user. Most of the input can be determined relatively accurately with the exception of soil damping constants. One goal of this study is to give the reader guidelines for determining the soil damping constants of different California soils. The use of soil damping constants in WEAP requires considerable engineering judgment. Besides, there are two choices of soil damping constants. Smith's damping constant is based on soil resistance(13). The Case damping constant is based on the pile properties(3).

From the use of data obtained from force transducers and accelerometers attached near the pile top during pile driving plus provided input parameters, the PDA calculates a static bearing capacity. Unfortunately, the soil damping constant is also a PDA input parameter. By defining a pile failure criterion and using data from pile load tests in comparison with PDA computed pile capacities, damping constants can be computed for use in wave equation programs. Soil setup can be accounted for by restriking the pile.

The overall objective of this study is to provide a method for developing useful qualitative data for determining Case Method soil damping constants. This paper attempts to relate the soil damping constant to a measurable engineering soil property. This measurable property is the standard penetration test value. In addition, this paper will familiarize the reader with the WEAP Program to the extent that she or he will be able to provide reasonable correct data for input into WEAP in order to yield answers comparable to field results. This study is only concerned with open end diesel hammers, and piles which are uniform in material and cross-section along their entire length.

CHAPTER 2

BACKGROUND

A general knowledge of wave mechanics will be useful in understanding how the wave equation is used by PDA. In this chapter, basic wave mechanics and calculation procedures by PDA are discussed.

A. Basic Wave Mechanics

When a pile with a constant cross-sectional area is subjected to an axial force, a stress wave is generated which travels to the pile tip with a velocity, c. The wave is then reflected from the pile tip back up to the top of the pile. The following sign convention will be used for the directions of the stress waves(4).

Sign	Force	Velocity
positive	compression	downward
negative	tension	upward

The speed with which the stress waves travel through the pile is given by the equation (5).

$$c = (E/p)^{1/2}$$

where

c = velocity of propagation of the stress wave

E = modulus of elasticity of the pile

p = mass density of pile material

The particle speed, V, (which differs from the wave speed, c), is the velocity with which a particle in a pile moves as the wave passes by(4).

For an unsupported linearly elastic rod (or pile as the case may be), it can be shown that the force (F) due to a sudden impact at one end of the rod is related to the particle velocity (V) by the following equation: (4)

$$F = VZ$$

where

$Z = EA/c$ (Z is referred to as the rod's impedance)

and

A = cross-sectional area of the rod

The one-dimensional wave equation can be derived from Newton's Second Law as follows: (5)

$$\frac{\partial^2 u}{\partial t^2} = \frac{E}{P}\frac{\partial^2 u}{\partial x^2}$$

where $\frac{\partial^2 u}{\partial t^2}$ represents acceleration and $\frac{\partial^2 u}{\partial x^2}$ represents the strain gradient at time, t, and location, x. The solution to this equation describes the way a wave travels in a pile or rod that is free at both ends, and can be expressed as (5)

$$u(x,t) = f(x-ct) + g(x+ct)$$

where u is the displacement and f and g are functions which describe the shapes of the displacement waves that travel along the pile. The shapes may take any form and are determined by the boundary conditions.

In order to illustrate wave mechanics, three general cases of wave motion will be examined. They are as follows:

B. Case I: Pile Free at Both Ends(6)

This pile is free at both ends and there is no skin friction. For this case, force waves are related to the particle velocity in the wave as follows:

$V_d = F_d/Z$ subscript d denotes downward

$V_u = -F_u/Z$ subscript u denotes upward

When the pile top is hit by a hammer, a stress wave is produced (F_i) which travels to the pile tip with a particle velocity, V_i. Note that:

$$F_i = F_{id}$$
$$V_i = V_{id}$$

When the wave reaches the pile tip, the wave is reflected back up. At the tip, the wave changes from a (downward) compression wave to an (upward) tension wave. Due to the free pile condition, the force at the tip must be zero (force equilibrium) and the displacement at the tip is doubled. The result is:

$$F_{iu} = -F_{id} = -F_i$$

The particle velocity in the reflected wave remains the same as in the initial wave. Therefore,

$$V_{top} = V_{iu} + V_{id} = 2V_i$$

Similarly,

$$V_{bot} = 2V_i = 2F_{id}/Z$$

See Fig. 1 for the summary of force and velocity waves.

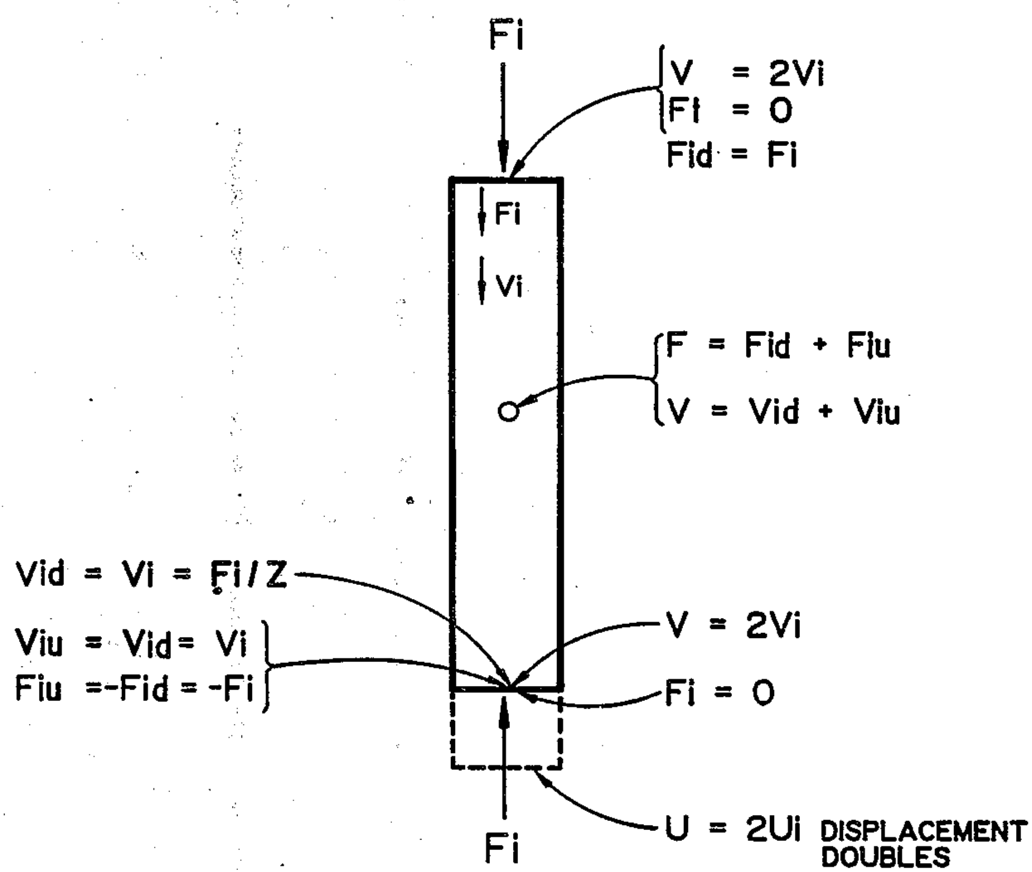


Fig. 1. CASE I; PILE FREE AT BOTH ENDS

C. Case II: Resistance at Tip(6)

In this case, the pile tip is fixed. The result is that the displacement and velocity at the toe are both zero, and the pile rebounds up. Therefore, at the toe we have

$$V = 0 = V_{id} + V_{iu}$$

or

$$V_{id} = -V_{iu} \text{ and } V_{iu} = -F_{id}/Z$$

At the pile toe

$$F_i = F_{id} + F_{iu}$$

and

$$F_{id} = F_{iu}$$

Hence, the force doubles at the toe

$$F = 2F_i$$

The force is zero at the pile top at time 2L/c since the pile is rebounding upward. L is the pile length. This results in a particle velocity of

$$V_{top} = -2V_i$$

See Fig. 2 for a summary of this case.

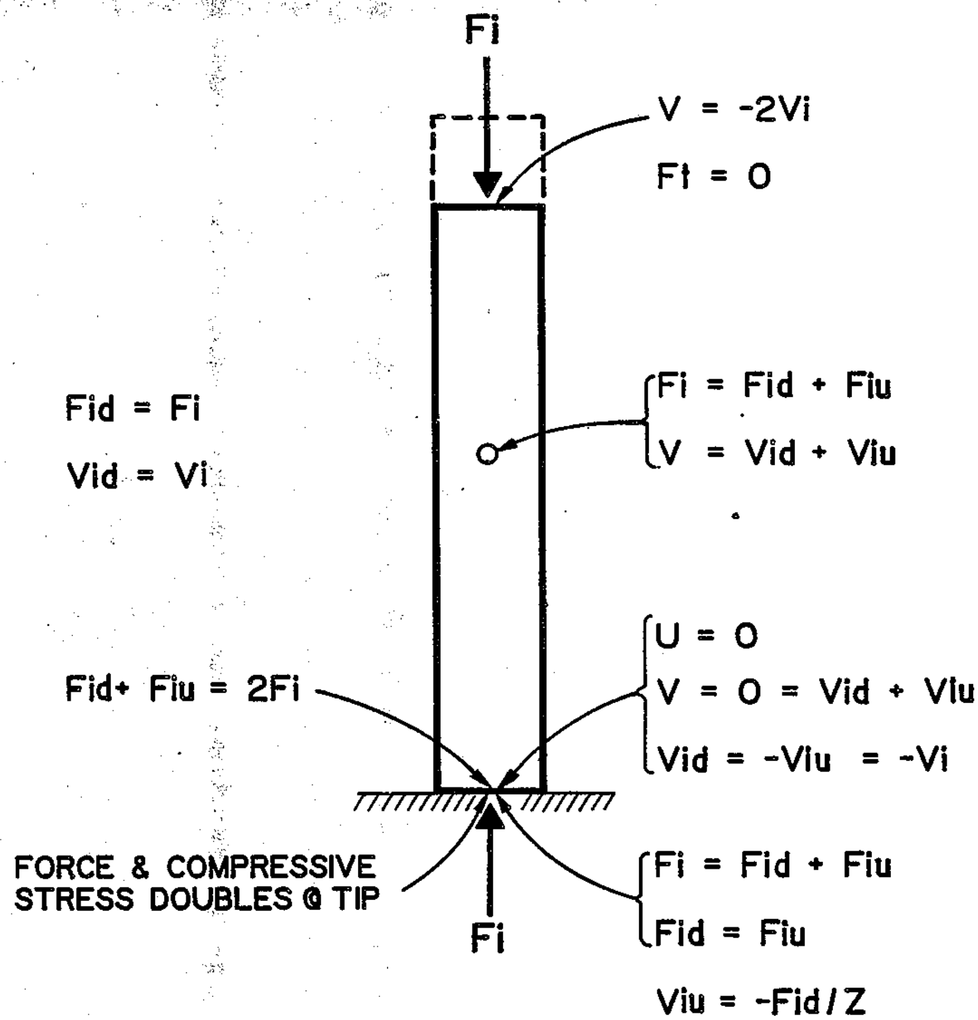


Fig. 2. CASE II; RESISTANCE AT PILE TIP

D. Case III: Resistance Along Pile Shaft(4)

For this case, an impact force at the pile top activates a resistance force at point x on the pile which begins at time t=x/c (see Figure 3). This wave travels upward initially, hence it is in compression. As in previous cases, the impact force also creates a downward tension wave. The magnitude of both waves is R/2. The particle wave velocity is given by

$$V = R/2Z$$

where

R = soil resistance

The upward wave reaches the pile top at time t=2x/c (see Figure 4). The tensile resistance wave reaches the pile tip at time t=L/c; at this point, it is reflected into a compression wave and returns to the pile top at time t=2L/c.

For a free pile top, the forces in the resistance wave have to cancel and the upward resistance waves are reflected downward in tension. This causes a doubling of the upward directed velocities. Therefore, the velocity at pile top is

$$V = R/Z$$

for $T<2L/c$

Figure 4 shows force and force-velocity wave traces for times t_1 and t_2 where,

$$t_2 = t_1 + 2L/c$$

and $t_1 = 0$

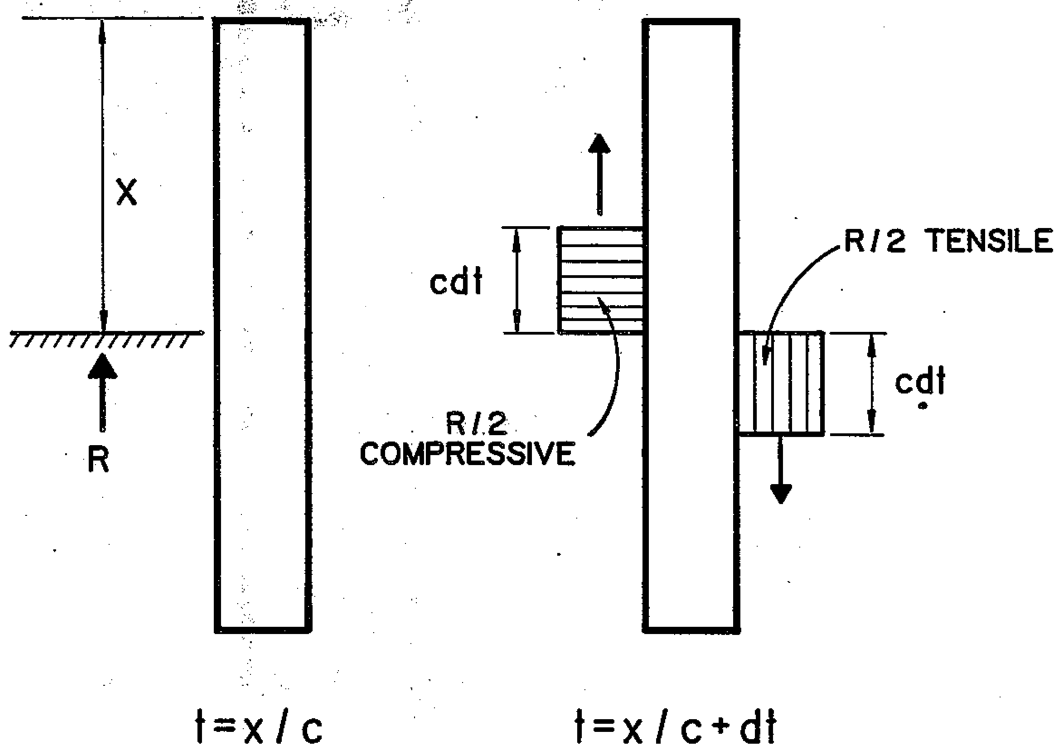


Fig. 3. CASE III, RESISTANCE ALONG PILE SHAFT (From Pile Dynamics Inc.)

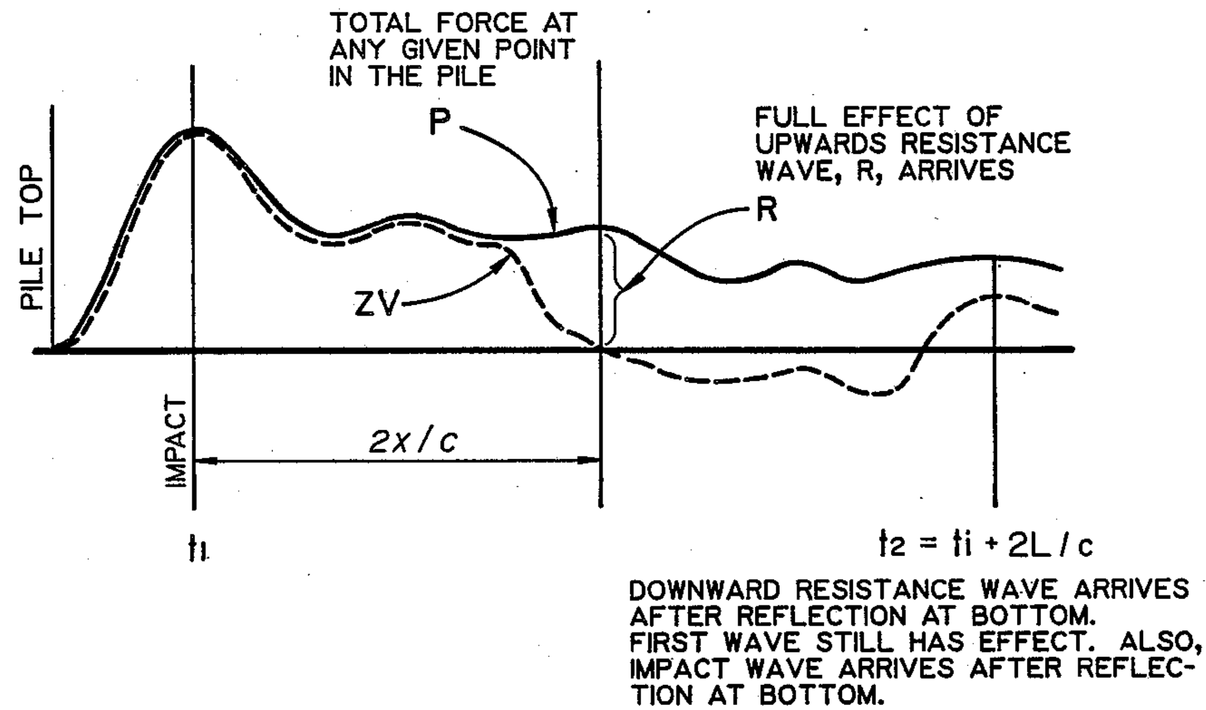


Fig. 4. FORCE TRACES (From Pile Dynamics Inc.) (4)

Assuming that the resistance force, R, is constant throughout the time period $x/c<t<2L/c$, then at time $2L/c$, the force and velocity records contain the effects of: (4)

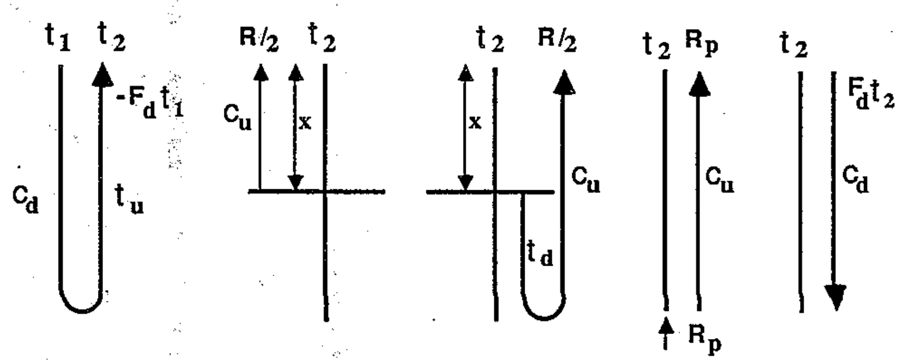
1. Upward traveling tension wave (t_u) which is due to the reflection (at the pile bottom) of the downward traveling compression wave (C_d) input at a time, $2L/c$, earlier ($-F_d t_1$).

2. Summation of all upward traveling compression resistance waves (C_u) ($+R/2$).

3. Initially downward traveling tension resistance waves (t_d) now traveling upward in compression after reflection at the bottom ($R/2$), and arriving at the pile top together with (1).

4. All downward traveling waves ($F_d t_2$).

The four-part effects can be shown as:



PART 1 PART 2 PART 3 PART 3 PART 4

The sum of these part effects is:

$$Pt_2 = -F_dt_1 + R/2 + R/2 + R_p + F_dt_2$$

where $Pt_2 = F_dt_2 + F_ut_2$ at any point on the pile

resulting in

$$F_ut_2 = R/2 + R/2 + R_p - F_dt_1$$

Waves parts 2 and 3 have a total value of R since both contain half wave skin friction and full-end bearing. Thus,

$$F_ut_2 = R - F_dt_1$$

the combination of all upward traveling waves contains the resistance and the bottom reflected (negative) impact wave of time, t_1.
rearranging,

$$R = F_ut_2 + F_dt_1$$

since $F_dt_1 = (P_1 + ZV_1)/2$ and

$$F_ut_2 = (P_2 - ZV_2)/2$$

$$R = (P_1 + ZV_1)/2 + (P_2 - ZV_2)/2 \quad \text{. . . . . Eq.1}$$

P which is also called F is the total force at any given point in the pile. R is the total resistance encountered during a complete passage of the wave for the period 2L/c. Determining the static capacity of the pile will require the following considerations: (4)

1. Eliminate effects of soil damping.

2. Proper choice of t, such that R is at full magnitude when P and V measurements are taken.

3. Soil setup or relaxation

4. Correction for R_s that decreases during 2L/c because of early pile rebound (negative velocity before 2L/c).

5. The pile must experience permanent set during the testing.

E. Soil Damping

Damping can be defined as a viscous parameter which is dependent on the physical characteristics of the soil. Material damping occurs when stress waves pass through the soil. Damping can also be thought of as a measure of the loss of energy in the pile resulting from soil hysteresis. In this case, energy is defined as the integral of the product of force and velocity for a time and hysteresis is the soil's ability to retard the energy within the pile. The damping effects of the soil and unloading of the pile due to rebound will combine to diminish the stress waves.

Most of the effects of damping are concentrated at the pile tip. In the Case-Goble Method, the damping force, R_d, is assumed to be proportional to the bottom velocity as follows: (4)

$$R_d = JV_b$$

$$R_d = J_c Z V_b \quad \text{.......................... Eq. 2}$$

where $J = J_c Z$

J_c is a dimensionless damping parameter.

The value of R_d is estimated during the first wave return period, 2L/c. The bottom velocity for a free pile condition after impact velocity arrives and reflects as shown in Case I is:

$$V_b = 2V_1$$

or $$V_b(t) = 2V_{top}(t - L/c) \text{ for } L/c <= t <= 3L/c \quad (15)$$

since measurements are made at the top.

For an actual pile, the effect of the downward traveling wave caused by the total soil resistance, R, on V_b is:

$$V_b(t) = -(c/EA)R \qquad \text{for } L/c <= t <= 3L/c \qquad (15)$$

The total bottom velocity is:

$$V_b = 2V_{top}(t - L/c) - cR/EA \quad (15)$$

recall that $R_d = J_c Z V_b$

we now have

$$R_d = J_c Z[2V_{top}(t - L/c) - cR(t)/EA]$$

$$R_d = J_c[2V_{top}(t - L/c)Z - R(t)]$$

since $R_s = R - R_d$ . . . . .Eq. 3

where R_s = static soil resistance

$$R_s = R - J_c[2V_{top}(t - L/c)Z - R(t)]$$

now, $$V_b = 2(F_d/Z)(t - L/c) - cR/EA$$

$$R_s = R - J_c[2F_d(t - L/c) - R(t)]$$

$$F_d = (P + ZV)/2$$

thus, $$R_s = R - J_c(P_1 + ZV_1 - R)$$

substituting R from Eq. 1 yields

$$R_s = 1/2\ (P_1 + ZV_1 + P_2 - ZV_2) - 1/2\ (J_cP_1 + J_cZV_1) - J_c/2\ (P_1 - ZV_1 - P_2 + ZV_2)$$

where $P_1 = F_1$ and $P_2 = F_2$

F. Summary, Case III

In summary, Case III provides the rational method for computing the pile static bearing capacity by the PDA. This is known as the Case-Goble Method. The static and dynamic resistances may be expressed as:

RTL = RSP + RD

where RTL = R total soil resistance

$RSP = R_s$ static soil resistance

$RD = R_d$ damping soil resistance

from Eq. 1

$RTL = 1/2\,[F_1(t_1) + F_2(t_2)] + EA/c[V_1(t_1) - V_2(t_2)]$. . . Eq. 1a

where $t_2 = t_1 + 2L/c$

$t_1 = t_{imp} + \Delta$

Δ = time delay constant

from Eq. 2

$R_D = J_c Z V_b$ . . . . Eq. 2a

where $V_b = 2(F_d/Z)(t-L/c) - cRTL/EA$ velocity at pile tip

$RD = J_c(P_1 + ZV_1 - RTL)$

from Eq. 3

$RSP = RTL - RD$ . . . . Eq. 3a

CHAPTER 3

This chapter describes the Pile Dynamics Inc. Model GB System for performing dynamic measurements. The equipment consists of the PDA, a tape recorder, and an oscilloscope. A general description of the equipment follows:

A. Tape Recorder

The recorder has four channels; one each for force, acceleration, velocity, and flutter reduction. The voice channel is combined with all four channels and must be turned off when recording data. Standard magnetic tapes are used for recording.

B. Oscilloscope

The oscilloscope has the capability of showing two traces on a time base. Normally, force and velocity (force) waves are viewed. The horizontal axis of the screen is the time base measured in milliseconds. The time base can be expanded with the PDA if necessary data is not shown on the chosen scale. The vertical axis, representing force, is measured in volts. The force amplitude can be adjusted by the y axis control on the oscilloscope.

C. Pile Driving Analyzer (PDA), Definitions, Printout

The analyzer converts electric signals collected from accelerometers and strain gages bolted to the pile and calculates forces and velocities. The following parameters should be defined before continuing further.

L ... Length of pile (ft)

L_g .. Length of pile below measurement point (ft)

m ... Total mass of pile below measurement point (lbs-ft/sec)

E ... Elastic modulus of pile (ksi)

A ... Area of pile (in^2)

c ... Pile stress wave speed (ft/sec)

F ... Force (kips)

V ... Velocity ft/sec

k_s .. Force transducer calibration (microstrains/volt)

K_a .. Accelerometer calibration 200 g's/volt

Acc . Acceleration (ft/sec^2)

g ... Gravity acceleration (ft/sec^2)

J_c... Case damping constant (dimensionless)

When monitoring pile driving by the Case Western Reserve University Method, the following parameters are dialed into the analyzer.

$10mc/L_g = 10\ EA/c$ Impedance (kips-sec/ft) (3)

$40L_g/c$ = Time (msecs).

$F = 203c\ K_s \times 10^{-6}$ (kips)

Acc = $3.0K_a$ for low g, and 0.6Ka for hi g (ft/sec^2)

Δ = 000 when time delay method is not used.

J_c = Case damping constant depending on soil type.

The print-out from the PDA consists of some of the following variables:

EMAX ... Maximum energy from integration of the product of force and velocity.

F_{max} ... Maximum measured compression force in pile at transducer location (kips).

V_{max} ... Maximum downward (ft/sec) velocity in pile at transducer location, calculated from integration of acceleration.

RTL ... Total soil driving resistance, static and dynamic (kips).

RAU ... Case-Goble Resistance, Automatic method, capacity when $V_b = 0$, $J_c = 0$, piles with very small skin friction.

RSP ... Case-Goble static resistance using J damping, RTL=RSP when J = 0.

RSU ... Case-Goble method for resistance accounting for early unloading long friction piles, measured velocity can go negative before $2L_g/c$.

RMAX ... J<.40 Case-Goble maximum resistance method uses RSP with damping. The time T_1 is automatically varied searching for RSP maximum. Do not use if TMX is equal to $2L_g/c$.

RMN ... Minimum Case-Goble resistance using damping, for situations where low blow counts, 40 blows/ft, can be used with confidence.

Wd, Wu ... Downward and upward traveling force waves at impact time. The wave down and up can be projected on oscilloscope screen to determine, skin friction, pile tension stresses, and pile damage.

DMX ... Maximum downward displacement of pile at transducer location per blow of hammer.

BPM ... Blows per minute of pile driving hammer.

TMX ...the delay time after the time of impact where the Case-Goble Method gives the maximum result, RMAX.

Only five variables can be printed out at the same time by the printer which is built into the analyzer. The selection of output variables can be changed at any time.

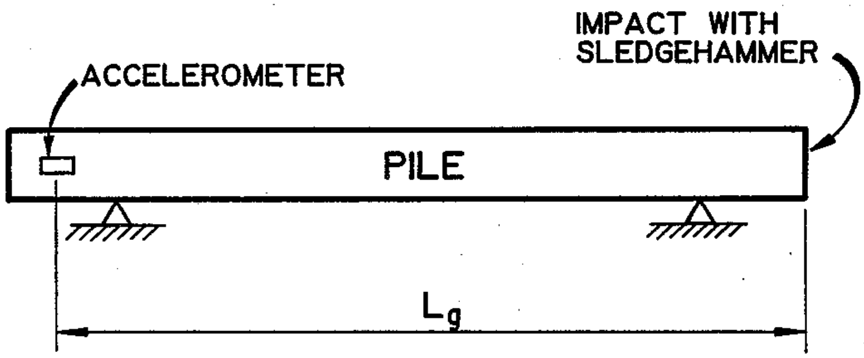
D. Instrumentation

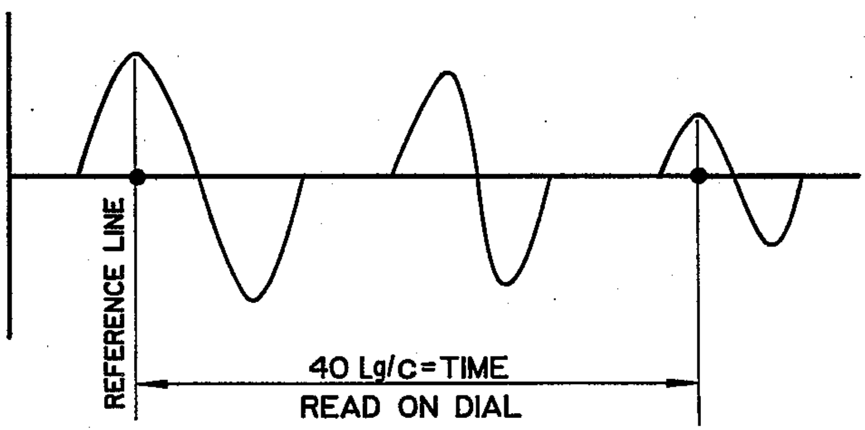
For concrete and steel piles, the force wheatstone strain transducers are mounted a minimum 2 feet below the pile top on opposite sides of the pile. The piezoelectric accelerometers are also mounted in opposite positions and on same level as strain gages. If possible, gages should be installed as low as the pile midpoint. This would prevent pile driving equipment from damaging gages during driving. Hi g accelerometers are used for steel piles and low g accelerometers for concrete and timber piles.

E. Pile Stress Wave Speed, c

The wave speed, as mentioned briefly in Chapter 2, is the speed with which a compression (or tension) zone moves along the pile. The wave speed "c" is a function of the material properties of pile. It is possible to determine wave speed from the velocity trace during driving. However, it is more reliable to determine the wave speed with a free pile.

The concrete pile wave speed is determined by installing an accelerometer near the end of a pile which is blocked off the ground. See Fig. 5. The opposite end of the pile is struck with a sledge hammer. The accelerometer registers a signal which, in turn, is accepted by the analyzer for conversion into velocity. It is recorded by the tape recorder and, on the replay, the velocity trace can be observed on the oscilloscope screen. Record at the





$$n \cdot \frac{40L_g}{c} = \text{TIME}$$

n = NUMBER OF PEAKS FROM REFERENCE LINE

Fig. 5. WAVE SPEED DETERMINATION

highest level of sensitivity by dialing 999 for Acc on the PDA and switch to Hi g. The time for the selected number of wave peaks can be determined from the $40L_g/c$ dial on the PDA. The dots on the oscilloscope screen define $40L_g/c$. Once time is obtained, the wave speed can be computed. Steel piles may use a wave speed of 16,800 ft/sec. Prestressed concrete pile wave speeds have been found in this study to vary from about 11,700 to 12,900 ft/sec.

F. F and V Relationship

The relationship between force (F) and velocity (V) should be explained to provide a better understanding of the wave equation procedure(6). As discussed earlier in this report, force is proportional to velocity.

Fmax = EA/c Vmax until a time after impact of less than $2L_g/c$.

This relationship can be derived as follows:

V ... Pile velocity ft/sec

t ... Time duration of hammer impact (msecs)

Δ ... Displacement of pile top (inches)

F ... Force or load on pile top (kips)

The displacement, Δ, at the pile top due to compression of a part of pile under a constant load is:

$$\Delta = FL/EA \quad \text{where } L = ct$$

If the load, F, is not constant with time, the displacement may be written as the integral

$$\Delta = \int_0^t F(t) \;.\; c \;.\; dt/EA$$

and for an increment of displacement with respect to time:

$$d\,\Delta/dt = F(t) \cdot c/EA = V(t)$$

resulting in:

$$F(t) = EA/c \cdot V(t)$$

or

$$F_{max} = EA/c \cdot V_{max}$$

Essentially, the above results tell us that when the pile top is struck by a hammer, the pile accelerates downward and the force in the pile is equal to the pile velocity, (V), times the dynamic impedance, (EA/c).

Two force traces are presented on the screen of the oscilloscope. They are F and EA/c • V. The first peak of the observed traces should be coincidental. If not, proportionality does not exist and input parameters should be examined for error. A common source of error is the wave speed, c, where the value is incorrectly measured or assumed. An accurate wave speed, c, is necessary for calculating the correct pile bearing capacity.

G. Distribution of Damping Constant, J_c

As discussed earlier in this paper, the J_c value is critical in calculating static soil resistance from force and force velocity waves. The calculated static bearing capacity decreases as J_c increases. Fig. 6 shows how the J_c value can be distributed during pile driving. A realistic damping distribution is shown on the left of Figure 6. The pile side distribution will most likely be more complex than that shown. The PDA assumes the Case-Goble J_c value is distributed at the bottom of the pile as shown on the right of Fig. 6.

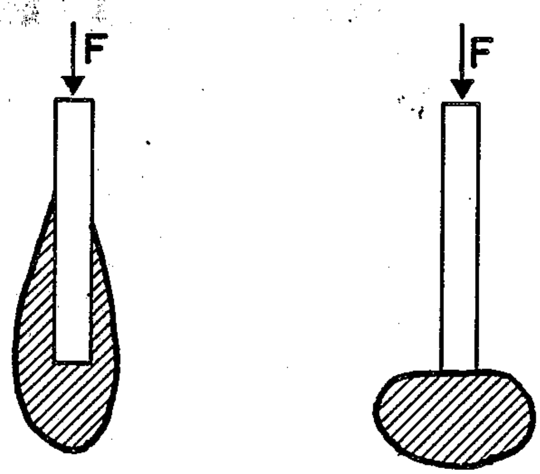


Fig. 6. SOIL DAMPING

H. Hammer Performance

Actual hammer performance can be monitored with the PDA. The energy transferred into the pile top can be measured with the PDA when the EMAX output parameter is selected. EMAX is really the integral of force times velocity for a small time increment(4). The stroke is calculated from measured blows per minute (BPM) of the hammer(4). The efficiency transfer ratio (TR) can then be calculated from the following equations if the ram or hammer weight, is known.

Stroke = $4.01(60/BPM)^2 - 0.3$ (Open end diesel hammers, ft)

Transfer Ratio = EMAX/(W) (Stroke) (where W is the weight of hammer, percentage).

CHAPTER 4

Dynamic measurements and results of static load tests are presented in this chapter. Also, the phenomenon of soil setup and its influence upon the results is discussed. The log of borings describing soil conditions at the sites is included in Appendix A. The log of borings contain data from standard penetration tests (SPT) using a 1.4 inch inside diameter split barrel sampler. The SPT values were used to develop a relationship between J_c, the soil damping constant, and the SPT value (N) for different sandy soils at the pile tip.

A. Test Sites

Fig. 7 shows the location of the ten California test sites. The test sites are listed as follows:

1. San Francisco Airport Route 101

   a. Off-Ramp Bent 15
   b. Off-Ramp Bent 12
   c. On-Ramp Bent 13
   d. 380 Northbound Viaduct Bent 21
   e. 380 Northbound Viaduct Bent 35

2. San Mateo 92/101 Interchange

   a. NW Connector Bent 21

3. Russian River at Jenner Rte 1

   a. Russian River Bridge Bent 6

4. Richmond Rte 580

   a. Bayview Avenue O.C. Bent 3

5. Oakland Rte 17 & 980

   a. Madison St. U.C. Bent 5
   b. Adeline St. Viaduct Bent 8

6. Newark Rte 84

   a. Newark Seal Slab Bent 25

7. San Jose Rte 87

   a. Bassett O.H. Bent 4
   b. Park Avenue Widen Abutment 3
   c. Guadalupe River Viaduct Bent 3

8. Sacramento

   a. Sacramento Light Rail Transit, Bent 4, 18th to 24th St. Bridge

9. Stockton Crosstown Viaduct

   a. Bent 29L

10. San Diego, Sweetwater River Bridge

    a. Abutment 1

In Appendix A, the soil conditions for each site are described, along with results of static load tests, and wave traces from dynamic measurements.

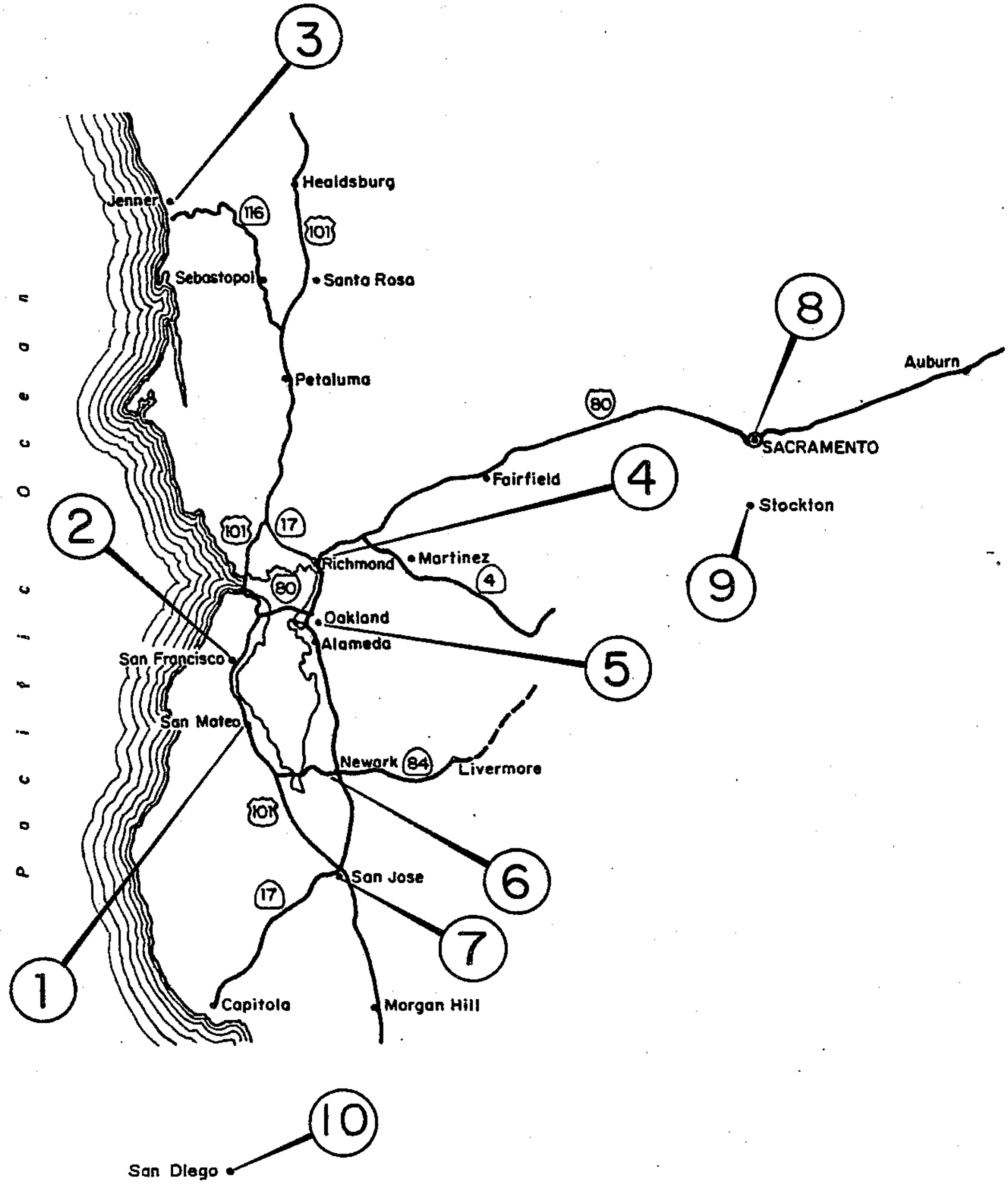


Fig. 7. TEST SITES

B. Static Load Tests

Static load tests were performed to confirm that the allowable pile head settlement at twice service load is within the range of acceptable values. If possible, the tests were taken to plunging failure without further increase in load. In the cases where this was not possible, a failure criterion had to be established. The defined failure was then used to determine the soil damping constant.

One popular criterion is Davisson's procedure(7). This procedure requires the evaluation of the pile elastic compression. In our opinion, the elastic compression cannot be evaluated to any degree of accuracy because of the effects of skin friction. The pile is not free standing as assumed. B.K. Hough proposed using half the pile length in calculating the elastic compression(1). The total settlement measured from the static load test is due to elastic compression of pile and movement of pile at the tip.

A pile has failed when the magnitude of settlement has reached a magnitude that can cause unacceptable cracking or difficulty with the structure. Generally, this magnitude is much smaller than plunging failure. The California State Department of Transportation (CalTrans) has chosen and implemented a total pile head movement of 0.5 inches at twice service load as an acceptable limit. This study uses that criterion.

For this study, all static load tests were conducted on piles that were to become part of foundations for highway structures. All piles were installed in a routine manner. Piles were loaded to plunging failure or to the limit of the testing equipment. The load was increased in stages, generally 10% of the double service load, with the settlement curve recorded at each stage of loading and unloading. The length of time for which each load increment was held was adjusted for the measured downward creep of the pile head. The final double service load was maintained for 60 hours.

C. Standard Penetration Test (SPT)

The SPT measures the soil resistance to sampler penetration. The split barrel sampler has an inside diameter of 1.4 inches and is 2 ft in length. The sampler is driven 18 inches into the soil and the last 12 inches is used to determine the number of blows per ft (N). The sampler is lowered into a bored hole large enough to prevent sidehole friction. A 140 pound drop hammer is used to deliver energy to the sampler via steel pipe and allowed to fall freely (30 inch free fall) until contact is made with the sampler. The number of blows necessary to penetrate the sampler 0.5 ft increments is recorded. The sampler is driven 1.5 ft, and the number of blows for the last 1.0 ft is recorded as the N value.

The driving of an SPT sampler can be considered a field test for the driving of a pile(10). Both involve hammer impact on a rod to produce a pulsed penetration into the soil. The penetration behavior is controlled by the stress wave movement in the rod and the dynamic resistance response of the soil. In 1976, Gallet performed a study utilizing a variation of Texas Transportation Institute (TTI) wave equation procedure to establish Smith's damping values(11).

TABLE 1 - Soil Damping Constants from SPT (secs/ft)

Soil Type	J_s	J_p
Sand	0.03	0.09
Clayey Sand	0.05	0.26
Clay	0.12	0.36

Gallet determined these damping constants for a number of SPT blows and obtained results within the range of values usually assumed in pile driving analyses. Soil damping, as it pertains to pile driving, can be considered to have two components, a side damping (J_s) and a point damping (J_p). Table 1 shows the magnitudes of the Smith's soil damping for a few soil types(11).

The disadvantage in using N values to correlate J_c values lies, in part, in the manner in which the test is performed and hence the amount of energy transferred to the rods. However, as Marcuson and Bieganousky observed, the value N is sensitive to changes in density, overburden pressure, and lateral stress conditions(12). These are the largest potential sources of data scatter. Other important variables that can affect the N value are:

1) use of drilling mud

2) diameter of hole

3) number of turns of rope on cathead

4) length and type of rods

5) variation in hammer drop-height

6) variation from the standard 18-inch penetration

7) rod setup due to remolding of soil

Despite the disadvantage, SPT is probably the most commonly used in situ test for geotechnical studies. For this study, testing conditions are more or less standardized by using the same drilling crews and equipment. Therefore, problems associated with repeatability and reliability have been minimized.

D. Soil Setup

Soil setup, the increase or decrease in skin friction along the pile sides with time has a significant effect on the soil damping constant(8). The substantial decrease of excess pore pressures in soils along the pile sides can cause an increase in pile capacity with time. Generally, soft sensitive clays will lose considerable shear strength during pile driving, but will regain all or part of their original shear strength with time. For saturated stiff clays, (N=10 to 20), the regain of strength as exhibited in pile capacity usually is incomplete. This is believed to be due to the creation of a gap between the stiff clay and the pile sides and remolding of the stiff clay(9).

For piles driven in silts, sands, and gravel, soil setup may have a lesser effect because of the relatively low skin friction along the pile sides and the lower generated excess pore pressures. However, a past history has shown cases where relaxation has reduced pile bearing capacity. Relaxation can occur if negative pore pressures develop during driving and dissipate with time(9).

Piles were driven through sands, silts and gravels at the Newark, Oakland, and Russian River sites. At these three sites, pile driving restrikes were not performed because they were tested early in this study and dynamic measurement restrikes were not included in the contracts. Fortunately, soil setup at these sites should not be a significant contributor to pile capacity. For example, a restrike for a Steel H pile in river pea gravel at Bent 6, Gianelli Bridge, Chico revealed a soil setup ratio of 1. A curiosity restrike next day of a bearing pile at Newark showed no increase in blows/ft.

For seven sites, San Francisco, San Jose, Richmond, San Mateo, San Diego, Sacramento, and Stockton; restrikes were included in the contracts. The upper soil layers penetrated by the piles at these locations were sensitive clays, except at Sacramento and Stockton. There, the upper soils consisted of compact silty fine sand.

Based on dynamically monitored restrikes made approximately 18 hours after initial driving, the following soil setup ratios were computed. See Fig. 8 and Table 2.

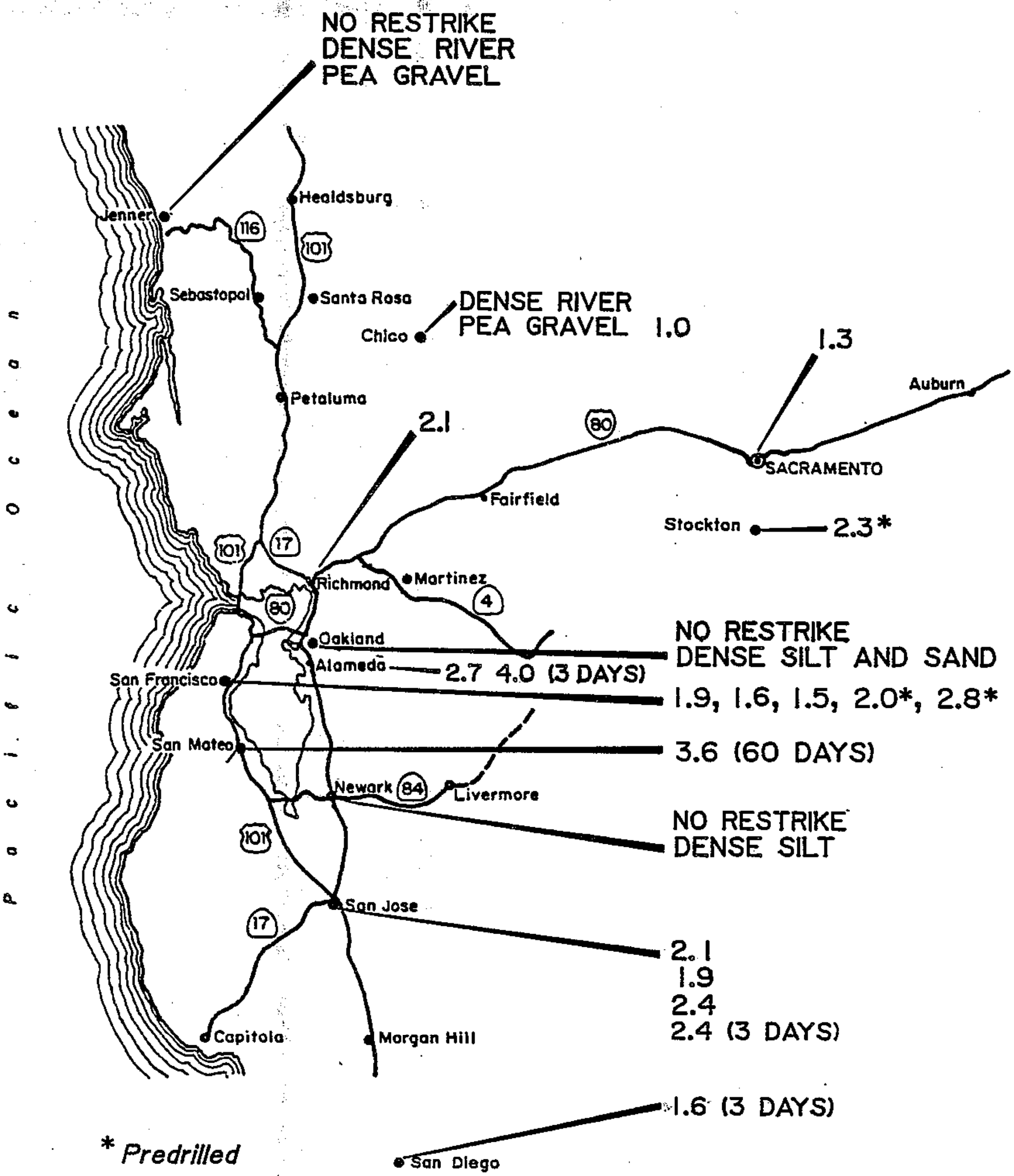


Fig. 8. SOIL SETUP RATIOS/FOUNDATION MATERIALS (After 18 hours)

TABLE 2 - Soil Setup Ratios

Site	Location	Setup Factor
S. F. Airport Off-Ramp	Bent 15	1.9
S. F. Airport Off-Ramp	Bent 12	1.6
380 Northbound Viaduct	Bent 21	1.5
380 Northbound Viaduct	Bent 35	2.8
S.F. Airport On-Ramp	Bent 13	2.0
Richmond, BayView O.C.	Bent 3	2.1
Alameda, Patton O.C.	Bent 20	2.7, 4.0 (3 days)
San Jose, Guadalupe River Viaduct	Bent 3	2.1
San Jose, Bassett O.H.	Bent 4	1.9
San Jose, Park Avenue U.C.	Abutment 3	2.4
San Jose, R2 Wall	R2 Wall	2.4 (3 days)
San Mateo, Northwest Connector	Bent 21	3.6 (60 days)
Sacramento Light Rail Transit	Bent 4	1.3
Stockton Crosstown Viaduct	Bent 29L	2.3
San Diego Sweetwater River Br.	Abutment 1	1.6
Chico, Gianelli Bridge	Bent 6	1.0

Dynamic Measurements

Dynamic measurements of force, acceleration, and velocity were recorded for initial and restrikes during pile driving at the previously mentioned sites. All values of force, acceleration, and velocity, including EMAX (maximum energy) were measured at the level of the force transducers and accelerometers. The gages were attached 2 to 3 ft from the pile top. The measured and computed values are assumed to be applicable to the pile top.

The PDA takes the measured F_1 and calculated V_1 at time t_1, and F_2 and V_2 at time t_2 along with input 10EA/c or 10mc/L_g, to compute a static pile capacity. It first computes RTL from Eq. 1a on page 17 with an assumed J_c. RD can be computed from Eq. 2a on page 17. RSP can then be calculated from Eq. 3a on page 17.

The PDA is capable of computing the static bearing capacity by several methods. They have been named previously as RMAX, RMN, RAU, RSP, and RSU. Three of the outputs are important to this study and will be described further. They are RMAX, RSU, and RSP. RSP is a computed static bearing capacity when a value supplied as input, is used in the calculation J_c. RMAX is the maximum bearing capacity that is possible with the RSP computational method. RSU is the bearing capacity computation applicable for cases where the velocity becomes negative before time $2L_g/c$ due to skin friction. This condition may occur sometimes for very long piles.

For this study, PDA output RMAX was selected because the results compared well with results from the static load tests. Also, RMAX is a valid computational method when the J_c value is less than 0.4, as discussed in Chapter 3. RMAX uses the same equations as RSP. Using the selected $2L_g/c$ as a fixed quantity, the time t, is automatically varied and the resulting set of RSP values is searched for its maximum(3,4). The maximum RSP within these limits is thus reported as RMAX. TMX should always be checked to verify that the value is not equal to time $2L_g/c$.

F. Wave Traces

Examples of wave traces (from Appendix A) to illustrate some interpretation are shown in Fig. 9.

Trace "a" (Fig. 9) shows soft or easy pile driving when the second velocity peak is large. Note the good proportionality at the first peak as discussed on page 23. And, note that the traces diverge late. This is due to predrilling the first 40 ft. This example is from S.F. Airport, Rte. 380 Northbound Viaduct, Bent 35.

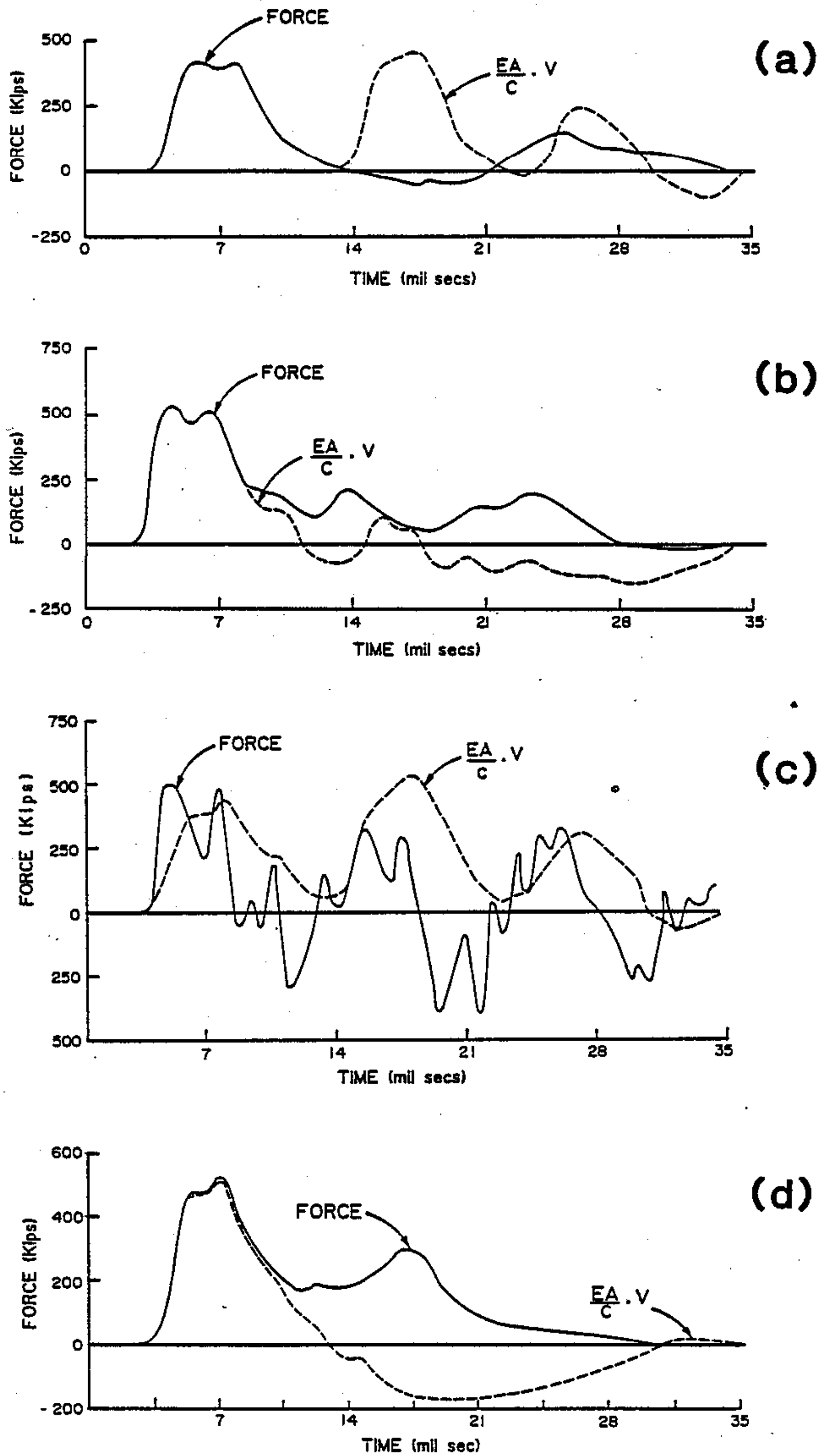


Fig. 9. WAVE TRACES

Trace "b" (Fig. 9) shows a restrike pile "a". Note again the good proportionality between the two traces at the first peak. And, note the smaller second velocity peak due to harder driving. The traces diverge earlier due to soil setup. The separation of the traces is an indication of soil resistance, as discussed on page 9.

Trace "c" (Fig. 9) is from Rte. 380 Northbound Viaduct, S.F. Airport, Bent 21, and shows what can happen when the lead anchors inserted in drilled holes in the concrete pile become loose during pile driving. The force transducers experienced uncontrolled vibration.

Trace "d" (Fig. 9) shows a pile restrike from San Jose, Park Avenue Widen, Abutment 3. The proportionality is very good at the first peak. The trace shows negative velocity before time $2L_g/c$. Therefore, RSU should be examined for bearing capacity along with RMAX.

G. Cushions and Followers

For this study, plywood cushions were used for pile driving. The cushions were 5 to 5-1/2 inches in thickness with a shape identical to the pile cross section.

Followers were not used for driving piles for the static load tests. Followers are used to drive piles cast with dowels sticking out of the pile top. The anchor piles were cast with a coupler flush to the pile top. This way a dywidag high strength steel bar can be screwed to the coupler. The steel bars hold the block beams that support the main steel beam for the static load test.

H. Soil Damping Constants vs SPT Values

Results of the dynamic test were correlated with results from static load tests to establish soil damping constants. This was done by adjusting the J_c input variable on the PDA until a match was made between the appropriate PDA computed static resistance and the value determined by a pile static load test. The damping constants arrived at in this manner were plotted against SPT values for the soils at the pile tips (Fig. 10). The J_c values and the boring logs which detail the SPT values are in Appendix A. In determining the J_c values, some engineering judgment was used since many of the static load test results did not reach our criterion of 0.5 inch settlement. This was due to equipment and anchor pile uplift capacity limitations. Since static load tests were completed a measurable amount of time after pile driving, the Fig. 10 plot includes the effects of soil setup. A complete evaluation of the temporal characteristics of soil setup was not within the scope of this investigation.

The SPT values, N, and soil description at pile tip for all sites are listed in Table 3: The sites at Chico (Gianelli Bridge) Alameda (Patton O.C.) and San Jose (R2 Wall) have been omitted because static load tests were not performed.

TABLE 3 - Site Soil Description and SPT

	Site	N at Pile Tip Blows/ft	Soil Description
a.	San Mateo NW Connector Bent 21	23	Very stiff clay and dense fine sand.
b.	S. F. 380 Northbound Viaduct Bent 21	33	Compact gray silty to clayey sand.
c.	S. F. 380 Northbound Viaduct Bent 35	48	Dense clayey silt.
d.	San Jose, Park Avenue Widen Abutment 3	24	Compact silt and silty fine sand.
e.	San Jose, Bassett St. OH Bent 4	24	Compact clayey silt.
f.	San Jose, Guadalupe Conn. Bent 3	18	Compact clayey sand.

TABLE 3 (Continued)

	Site	N at Pile Tip Blows/ft	Soil Description
g.	Richmond, Bayview O.C. Bent 3	23	Compact fine sand and stiff silty clay.
h.	S. F. Airport Off-Ramp Bent 12	25	Compact silty sand.
i.	S. F. Airport Off-Ramp Bent 15	27	Stiff sandy clay and compact clayey sand.
j.	S. F. Airport On-Ramp Bent 13	29	Compact silty sand and stiff sandy clay.
k.	Newark Seal Slab Bent 25	56	Dense coarse sand and gravel.
l.	Russian River Tip-45 Bent 6	70	Very dense coarse sand and gravel.
m.	Russian River Tip-50 Bent 6	77	Very dense coarse sand and gravel.
n.	Oakland, Adeline Street Bent 8	200	Very dense sand.
o.	Oakland, Madison Bent 5	100	Very dense sand.
p.	Sacramento Bent 4	31	Compact silty fine sand
q.	San Diego Abutment 1	42	Dense silty fine sand
r.	Stockton Bent 29L	40	Dense Cemented silt

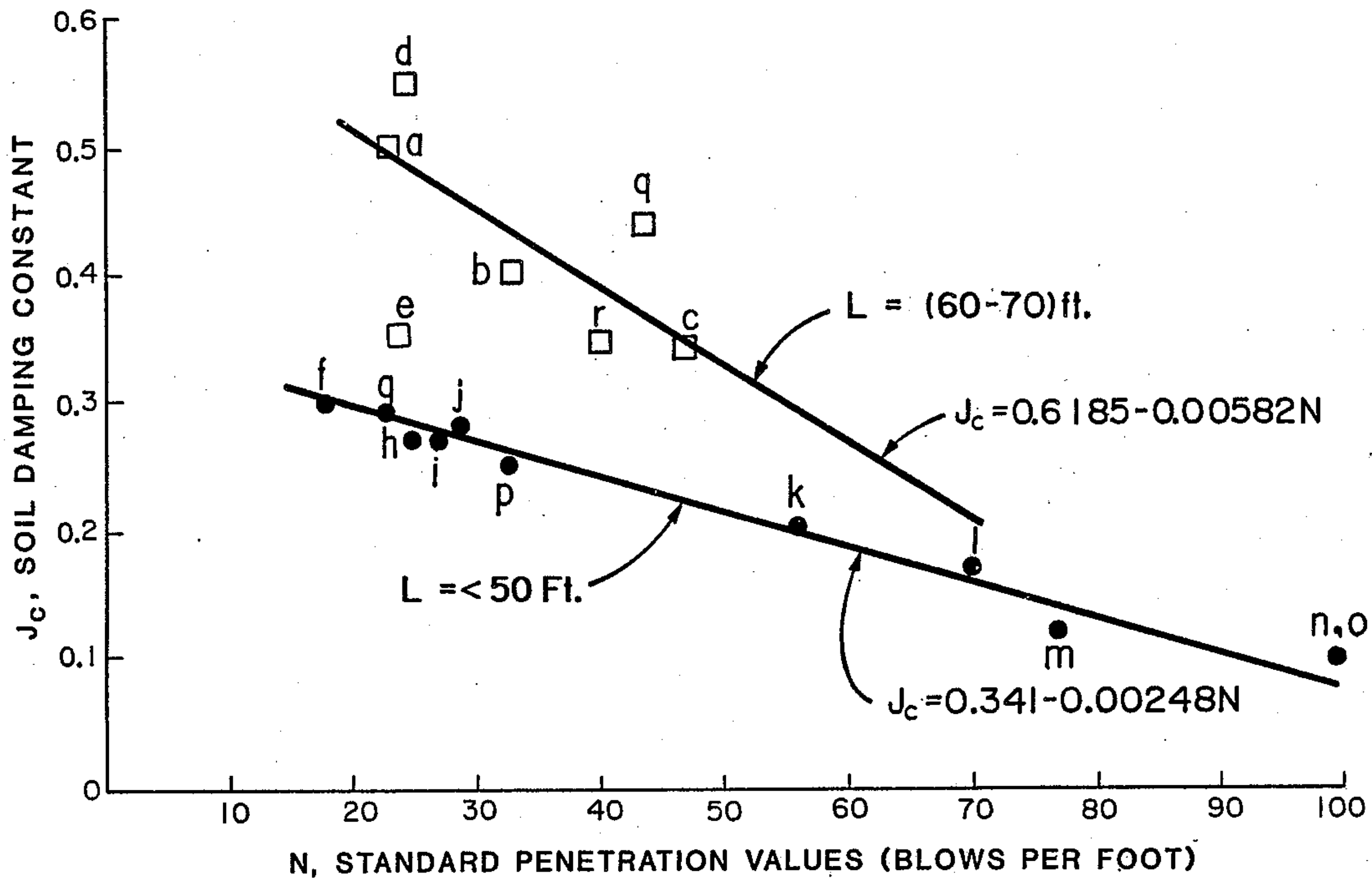


Fig. 10. CORRELATION BETWEEN J_C AND N

H. Discussion

Figure 10 shows two straight line fits by regression analysis of data points. The lower line is for piles with penetration lengths less than 50 ft. Actually, most of the lengths are between 36 to 45 ft. The upper line is for piles with penetration lengths greater than 60 ft and less than 70 ft and developed from points a, b, and c. Points d and q are 79 and 82 ft. and r is 56.3 ft. The straight line fit for piles less than 50 ft is well defined. The only point in question is j. The force and velocity proportionality was about 23%. A correction for wave speed could not be made because the recorder malfunctioned. The data points were calculated with static load tests using the Caltrans pile failure criterion. The double design load was often reached about 8 to 12 hours after the start of loading and held for 60 hours to monitor pile creep.

The upper straight line fit is inconclusive due to lack of sufficient data points. Also, data point (b) may be unreliable due to loose force transducers during pile driving. The lead anchors inserted in the concrete pile became loose during driving. However, the trend indicates that piles with longer penetration lengths will have higher soil damping constants. Longer piles with greater penetration lengths, in which skin friction plays a more predominant role, are difficult to quantify into a single J_c vs N curve. It also appears that for N greater than 90, pile lengths have little influence on the J_c value. Results from driving piles into very dense strata suggest a minimum recommended value for J_c of 0.10. Although two of the longer piles were predrilled 40 ft, soil setup did occur on pile restrike. Thus, the predrilling may have had an insignificant effect on the J_c value.

As discussed in this chapter, data points represent soil setup time of about 18 hours with the exception of point a (San Mateo NW 21), where the soil setup time was 60 days. Point e from San Jose Bassett St. Overhead falls in the category of a pile length greater than 60 ft, but did not fit the straight line. A large portion of the soil setup occurred after 18 hours and this apparently has rendered point e invalid. For example, a one-day and three-day dynamically monitored pile restrike in Alameda in soft to stiff clay showed soil setup continuing to increase after three days. The Alameda site is not part of this report. It appears that in most cases, the portion of the soil setup occurred within 18 hours. Points f, g, i, and 1 all failed by plunging or exceeding the 0.50 inch settlement criterion established for this study. These data points offer strong evidence that the J_c values determined for the lower line are reasonably correct.

Static load tests were performed 1 to 13 days after the restrike. This time lapse may have had an effect on the static pile capacity due to the occurrence of additional soil setup. Therefore, the J_c values determined from restrikes by comparing results from RMAX, RSU, and static load tests may be slightly in error. A slow rate of soil setup may lower the determined J_c values, since the static bearing capacity may continue to increase a significant amount after the PDA monitored restrike. Thus, the determined J_c values offered in this report would be conservative for cases where a large portion of the soil setup does not occur by the time of the PDA monitored restrike.

CHAPTER 5

In this Chapter, WEAP procedures are described and discussed. A limited parametric study employing the WEAP program and using damping constants determined from the PDA and static load tests is presented.

A. WEAP Program

WEAP (Wave Equation Analysis of Piles) is a computer program prepared by G. G. Goble and Frank Rausche of Goble & Associates under a Federal Highway Administration contract in 1976 (an updated version of the program was written in 1980)(2). The program documentation, entitled Wave Equation Analysis of Pile Driving, consists of three volumes: Background (Vol. I), User's Manual (Vol. II), and Program Documentation (Vol III). WEAP can be used to predict, prior to pile driving, the static pile capacity as a function of blow count per foot under the hammer.

While driving a pile, a hammer must act against both dynamic and static pile resistance. When in service, only static resistance remains as support offered by the pile. If it is desired to measure the static resistance of a pile from information gathered during driving operations, dynamic resistance must somehow be accounted for.

Several methods exist for determining static pile capacity; this topic, beyond its relevance to the WEAP program, is not within the scope of this report. Instead the emphasis is to determine proper input values to be used by the WEAP program in order to adequately model the dynamic forces encountered during pile driving.

The basic approach of a wave equation analysis is to separate the piling system into a series of rigid masses. In effect, this is a numerical approximation of the mass continuum that makes up the piling system (see Fig. 11). In order to simulate pile stiffness, the masses are connected to one another by a series of weightless springs. In a similar manner, internal pile damping is modeled with a dashpot between masses. The soil- pile interaction at the sides of each element, as well as at the pile tip, is also modeled with a dashpot and spring to simulate dynamic and static soil resistance, respectively.

The first wave equation analysis of pile driving suitable for a computer solution was synthesized by E.A.L. Smith in 1960 (13). Smith's numerical solution was expanded by Lowery, Hirsch and Samsom in 1967 and later developed by this group into the TTI computer program (14); this program was developed primarily for modeling piles driven with air steam hammers. WEAP uses a refined version of this technique and is capable of modeling both air steam and diesel hammers. This report, however, is only concerned with open end diesel hammers.

The WEAP program is similar to Smith's solution in that the pile driving model consists of hammer, capblock, cushion, pile and soil. The main difference between WEAP and the TTI program is the manner in which the soil damping is accounted for. TTI uses Smith's damping method, while WEAP can use either Smith's damping or another approach called Case damping. In both methods soil damping is proportional to the velocity imparted on the pile by the hammer. However, Smith's damping defines the proportionality in terms of static soil resistance, whereas Case damping defines the proportionality in terms of pile properties. Many authors believe that there is no rational reason for linkage between dynamic and static soil resistance (17). This is why the Case Method has been accepted by many to be a better approach to account for the effects of soil damping.

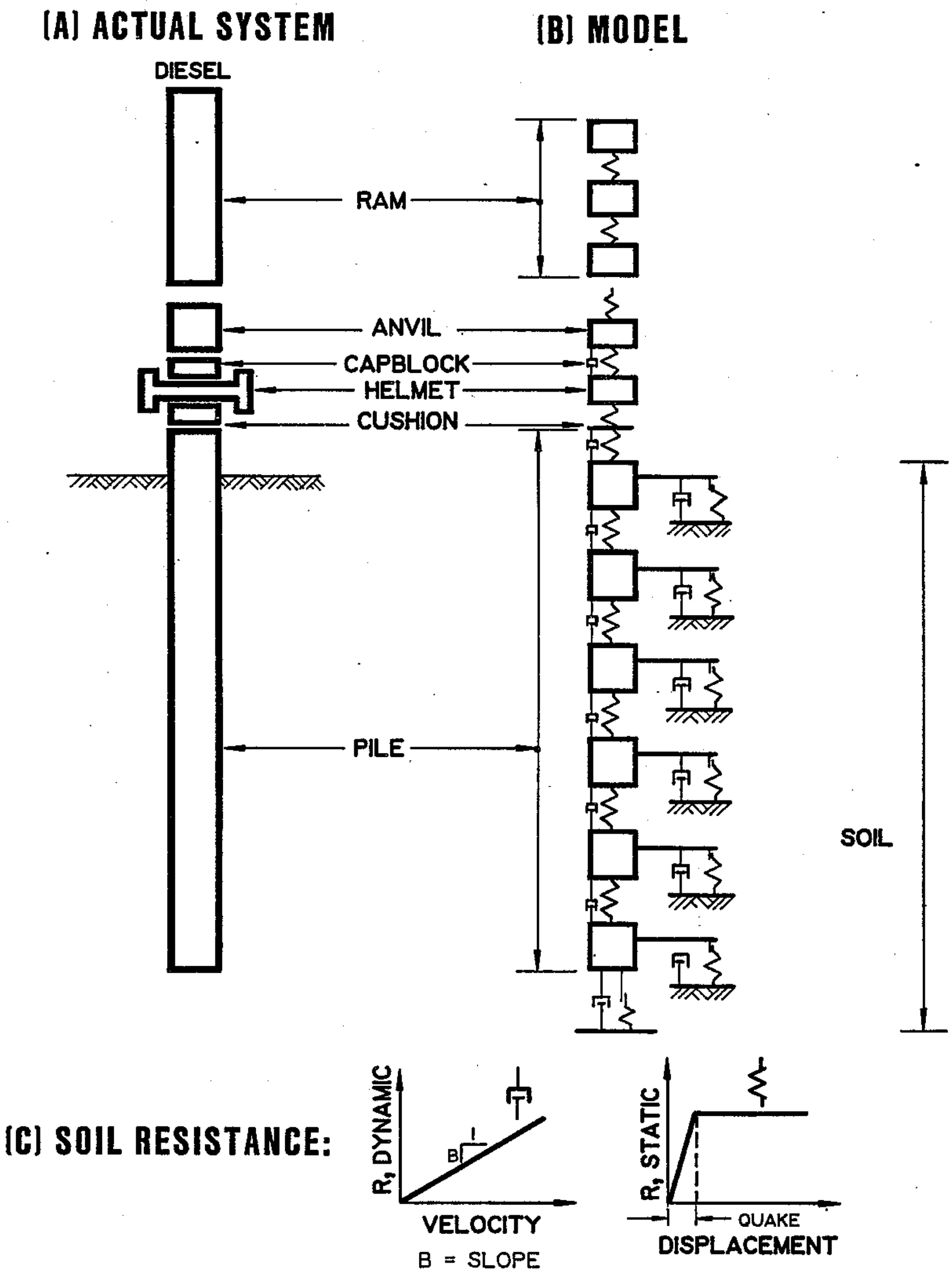


Fig. 11. (A) THE SYSTEM TO BE ANALYZED
(B) THE WAVE EQUATION MODEL AND
(C) THE COMPONENTS OF THE SOIL RESISTANCE MODEL (2)

1. Soil Resistance

In the WEAP program, static soil resistance for each pile element is assumed to increase linearly with displacement of the element. Resistance increases in this fashion until a displacement known as the quake is reached. Any displacement past the quake is assumed to occur under a constant resistance (see Fig. 11c).

When the element is unloaded, rebound is assumed to be equal to the quake. Therefore, the permanent displacement, or set, of the element is equal to the plastic displacement that occurred during loading (see Fig. 12). The energy absorbed in this process is equal to the set multiplied by the ultimate static resisting force (R_{ult}) the element is subjected to.

Also in Fig. 12 a more realistic force displacement plot is compared to the model. Three things should be noted in the comparison. First, the set in both cases is the same. Second, the ultimate static resistance for both cases are equal. Finally, the total area under each plot (dissipated energy) is about the same.

The WEAP program assumes the dynamic soil resistance for each pile element to vary linearly with the velocity of the element (see Fig. 11c). The dynamic resistance of an element is given by the equation:

$$R_d = BV$$

where V = element velocity

$B = J(EA/c)$ using Case damping.

J = Case damping factor associated with the pile element

E = Young modulus of the pile element

A = Cross sectional area of the pile element.

c = Stress wave velocity within the pile.

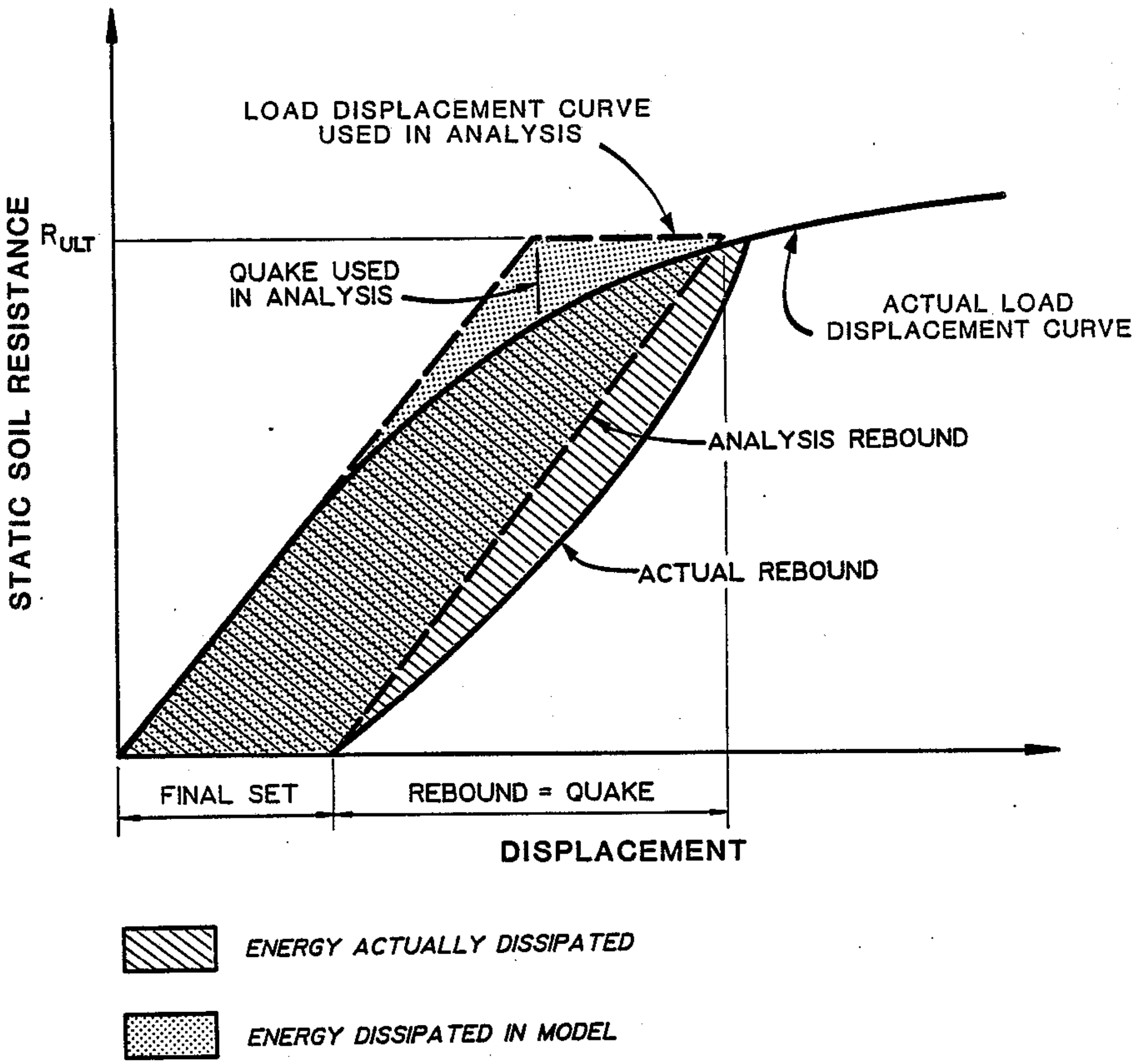


Fig. 12. REAL LOAD DEFLECTION CURVE & ITS MODEL. (2)

Some researchers have shown the assumption of dynamic resistance being proportional to velocity to be incorrect for clay soils(16). However, according to Authier & Fellenius(17), the assumption of linearity is considered acceptable for most practical cases.

Shown in Fig. 13 are the soil forces imparted to a pile element due to a single hammer blow. Also shown in Fig. 13 is a plot of time versus element displacement. It can be seen that R_d increases as the element velocity increases up to point A. After point A, an inflection point in the time-displacement plot, the element begins to slow down, therefore, there is a corresponding decrease in R_d. At point B, the element displacement has reached the quake, therefore, displacement occurs under a constant R_s and the total resistance begins to decrease due to the decrease in R_d. At point C, the element reverses direction (it is now moving upward) in an effort to dissipate stored energy, therefore, R_d becomes negative while R_s decreases. At point D, R_s reaches zero, however, momentum keeps the element moving in the upward direction. The element now acts as if it is being driven in the upward direction. The soil offers resistance to movement in this direction (a negative R_s) which, in turn, slows the element down, thereby reducing R_d in absolute terms. This cycle continues until movement is completely damped out.

2. Program Operation

The typical user, from the results of a static analysis, will specify the total ultimate pile capacity (R_{ult}) which is to be analyzed. The variable IPERCS (% skin friction) is used to specify what portion of R_{ult} is to be carried by pile skin friction. The variable ITYS is used to indicate how the skin friction is to be distributed along the length of the pile. For example, assume the user specifies a 30-foot pile, with an R_{ult} equal to 100 tons, IPERCS equal to 70%, and ITYS equal to 1 (triangular skin friction distribution).

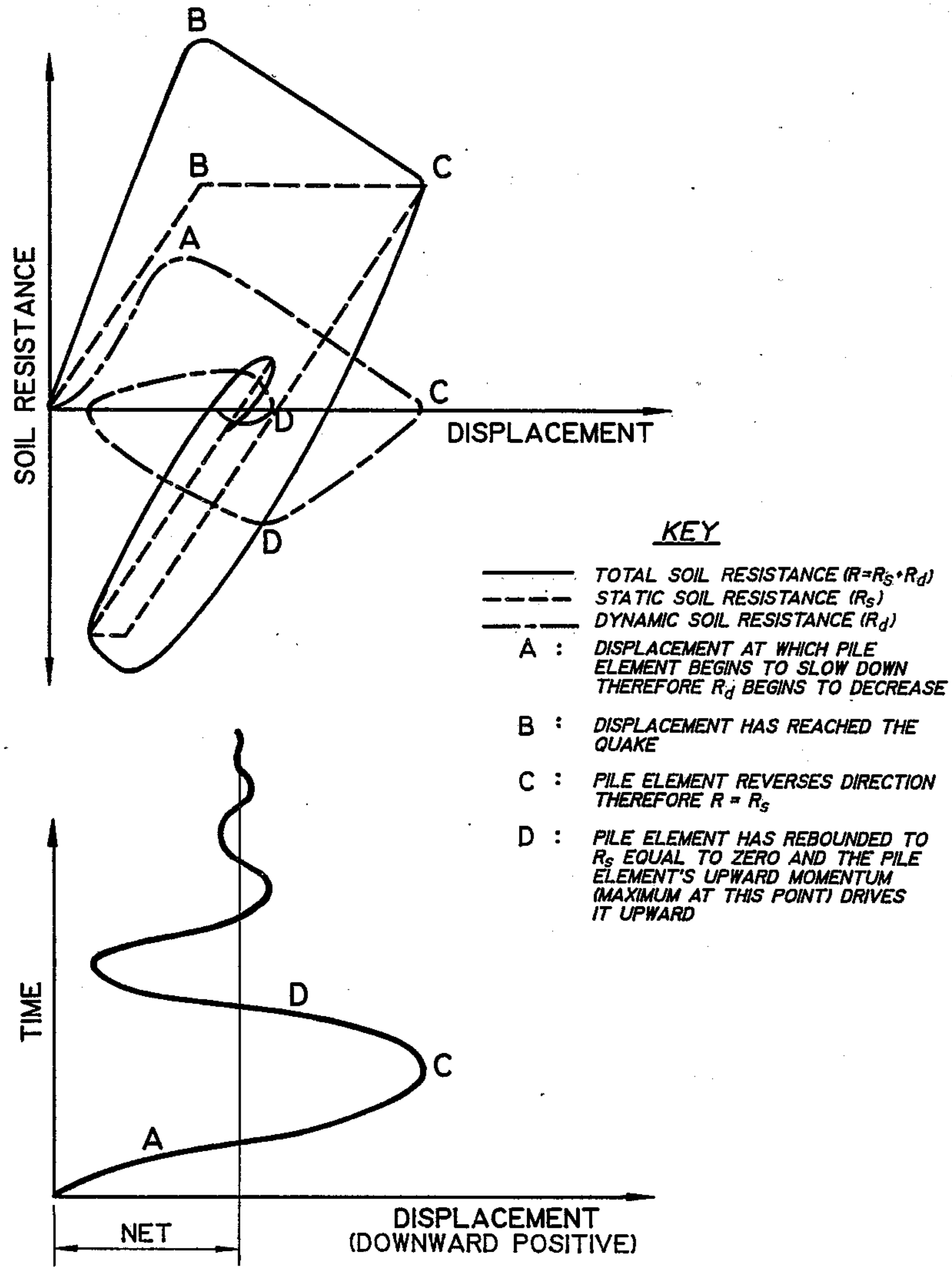


Fig. 13. MODEL USED FOR STATIC, DYNAMIC & TOTAL RESISTANCE (17)

WEAP would then break the pile into elements; in this case, assume the pile is broken into six 5-foot long elements as shown in Fig. 14. Also shown in Fig. 14 is the user specified skin friction distribution.

The total area under the skin friction distribution represents the percentage of R_{ult} contributed by skin friction (IPERCS), in this case 70 tons. That is, if the shaded area in Fig. 13 is X_{total}, then X_{total} = IPERCS.

Each element has a portion of X_{total} associated with it, we will call this area X_i. Therefore, the ultimate static skin resistance of the ith element is given by the equation:

$$R_{ult,skin}(i) = R_{ult}\ (\text{IPERSCS}/100\%)\ (X_i/X_{total})$$

Using the example, consider the 4th element:

$$X_{total} = (1/2)m(30\ \text{ft.})^2 = m(450\ \text{ft}^2)$$

where m = slope of skin friction distribution

therefore: $X_4 = (15')(m)(5') + (1/2)(m)(5')^2 = m(87.5\ \text{ft}^2)$

$R_{ult,skin}(4) = 100\ \text{ton}\ [70\%/100\%]\ [m(87.5)/m(450)]$

$R_{ult,skin}(4) = 13.6\ \text{tons}$

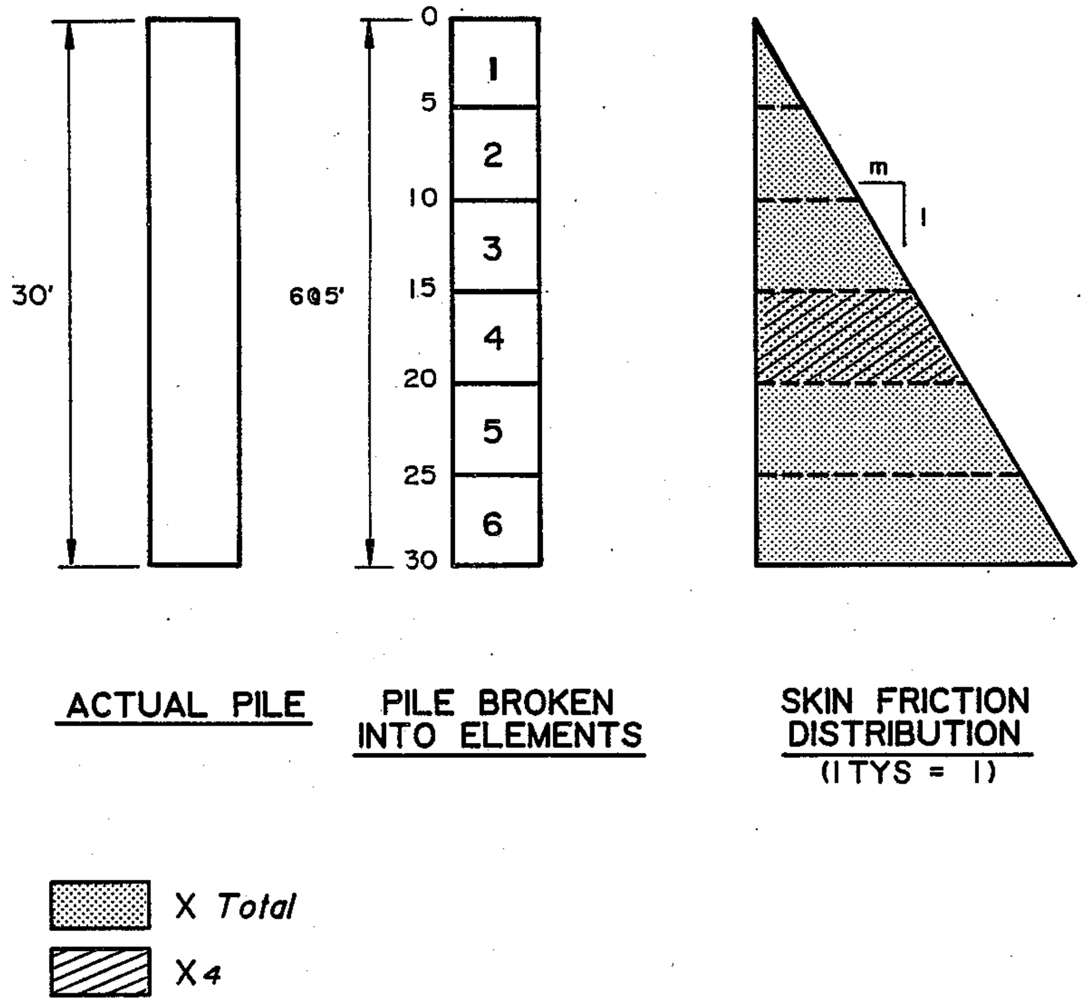


Fig. 14. PILE ELEMENTS & SKIN FRICTION DISTRIBUTION

The bottommost element must also take the forces due to toe bearing. In this example, the ultimate static toe resistance acts upon the 6th element and is given by the equation:

$$R_{ult,toe} = R_{ult}(100\% - IPERCS)/100\%$$

$$= 100 \text{ tons } (100\% - 70\%)/100\% = 30 \text{ ton}$$

The WEAP program uses the specified toe quake together with $R_{ult,toe}$ to determine the static resistance-displacement function for the bottommost element. The skin quake for each element is taken as the user specified skin quake (i.e., specified skin quake = squake(i) = squake(i + 1) = ...).

In order to specify the damping forces (dynamic resistance) the pile must overcome during driving, the user inputs two Case damping factors; J_{toe} and J_{skin}.

The damping factor for toe damping (J_{toe}) is used directly to determine the slope (B) of the $R_{d,toe}$ versus velocity line (see Fig. 11), i.e.:

$$B_{toe} = J_{toe}\, E_{toe}\, A_{toe}/c$$

The skin damping factor (J_{skin}) specified by the user refers to total skin damping. It is distributed using the same skin friction distribution that is used to distribute static soil resistance (see Fig. 14). That is, $X_{total} = J_{skin}$. Therefore, the slope (B) of the R_d versus velocity line for the ith element is given by the equation.

$$B_i = [X_i/X_{total}]\, J_{skin}\, [E_i A_i]/c$$

where X_i is, as in the static resistance case, the portion of X_{total} associated with the ith element.

B. PARAMETRIC EVALUATION

To assess how changes in input parameters affect the WEAP program results, one must consider the different types of information WEAP requires and the uncertainty involved in specifying this information. Data used by WEAP can be divided into four categories:

1. Hammer properties
2. Pile cap and cushion properties
3. Pile properties
4. Soil - pile interactions

1. Hammer Properties:

Normally, hammer properties can easily be specified using the hammer data file supplied with the program. The user merely inputs an index number to inform WEAP which hammer is to be used.

It should be noted that WEAP defaults to a hammer efficiency of 95%. This is often incorrect; diesel hammer efficiency can be as low as 65%. Efficiencies around 75% were found to be the most common. Fig. 15 shows an example of how hammer efficiency influences the P-n chart.

To assess hammer efficiency, wave traces as generated by WEAP were compared to those measured by the PDA. The first portion of a wave trace represents the energy transmitted to a pile from hammer impact. Hammer efficiency was gaged by varying the efficiency input into WEAP until the first part of the WEAP-generated wave trace was in reasonable agreement with the first portion of the measured wave trace.

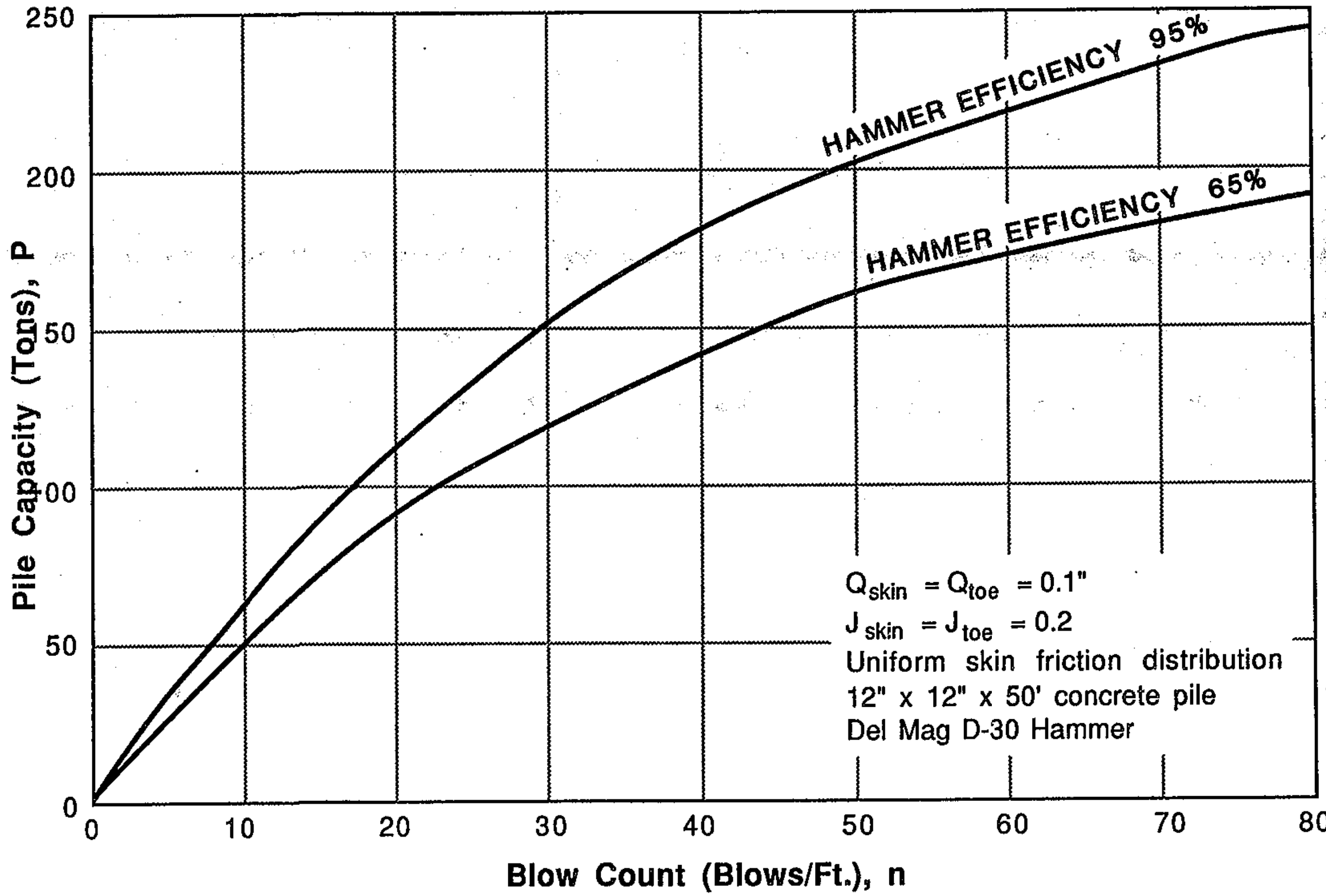


Fig. 15 - EFFECT OF HAMMER EFFICIENCY ON P-n CHART

It is typically not possible to quantify hammer efficiency when driving piles unless a PDA or similar instrument is available. Therefore, in order to be conservative, hammer efficiencies from 70% to 75% should be used as input for WEAP when generating P-n functions. When using WEAP to check possible pile overstressing, hammer efficiencies around 95% should be used in order to be conservative.

2. Pile Cap and Cushion Properties:

Pile cap and cushion properties can be determined provided the program operator is certain of the equipment the contractor will be using. In order to achieve this, it is suggested that the contractor be required to complete a form similar to the one shown in Fig. 16.

3. Pile Properties:

Pile properties can be easily specified because of the well known material properties of steel and concrete. For steel piles the modulus of elasticity (E) is taken to be 30,000 ksi. For concrete piles the modulus of elasticity was determined by measuring the stress wave speed (c) using the procedure outlined in Chapter 3, and back solving the wave speed equation mentioned in Chapter 2:

$$c = (E/p)^{1/2}$$

therefore:

$$E = p\ (c^2) = w\ (c^2)\ /\ g$$

Where g is the acceleration due to gravity, and w is the unit weight of concrete (taken as 150 pcf for all concrete piles).

Using this method, the average modulus of elasticity for the ten concrete piles studied was found to be 5000 ksi (rounding to the nearest hundredth). The standard deviation was 300 ksi.

Fig. 16. PILE DRIVING FORM

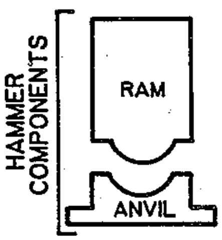
Contract No: ____________________ Structure Name and/or No.: ____________________

Project: ____________________ ____________________

____________________ Pile Driving Contractor or Subcontractor: ____________________

County: ____________________ ____________________

(Piles driven by)



HAMMER

Manufacturer: ____________________ Model: ____________________

Type: ____________________ Serial No.: ____________________

Rated Energy: ____________________ @ ____________ Length of Stroke

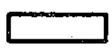
Modifications: ____________________

____

____

____

____



CAPBLOCK

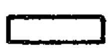
Material: ____________________

Thickness ____________________ Area: ____________________

Modulus of Elasticity - E ____________________ (P.S.I.)

Coefficient of Restitution-e ____________________

PILE CAP – Helmet / Bonnet / Anvil Block / Drivehead – Weight: ____________________



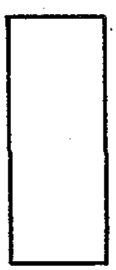
CUSHION

Cushion Material: ____________________

Thickness: ____________________ Area: ____________________

Modulus of Elasticity - E ____________________ (P.S.I.)

Coefficient of Restitution-e ____________________



PILE

Pile Type: ____________________

Length (in Leads) ____________________

Weight/ft. ____________________

Wall Thickness: ____________________ Taper: ____________________

Cross Sectional Area ____________ in^2 _

Design Pile Capacity: ____________ (Tons)

Description of Splice: ____________________

____

Tip Treatment Description: ____________________

____

NOTE If mandrel is used to drive the pile, attach separate manufacturers detail sheet(s) including weight and dimensions.

Submitted By: ____________________ Date: ____________________

4. Soil - Pile Interactions:

Soil-pile interactions are defined primarily by the following parameters:

- Qskin - The quake along the pile sides.
- Qtoe - The quake at the pile tip.
- IPERCS - The percentage of pile capacity that is developed by skin friction.
- Jskin - The Case damping constant of the soil along the pile sides.
- Jtoe - The Case damping constant of the soil at the pile tip.

Researchers (2, 18) have found that static soil resistance can be adequately modeled by using skin and toe quakes equal to 0.1 inch.

For production work, IPERCS is typically determined from the results of a static pile analysis. Fig. 17 shows an example of the effect IPERCS has on the P-n chart. Note that a relatively large change in the IPERCS results in only a marginal change in the WEAP generated P-n function. Therefore, error in IPERCS that results from inaccuracy in a static pile analysis will not have a significant effect on the WEAP generated P-n function.

The majority of the uncertainty in a wave equation analysis lies in the determination of proper Case damping constants. Fig. 18 shows an example of the P-n chart's sensitivity relative to the Case damping constant used in the analysis.

In order to aid in the determination of these damping constants, the Pile Dynamic Analyzer (PDA) was employed. Using the procedures outlined in

$Q_{skin} = Q_{toe} = 0.1"$

$J_{skin} = J_{toe} = 0.2$

Uniform skin friction distribution

12" x 12" x 50' concrete pile

Del Mag D-30 Hammer

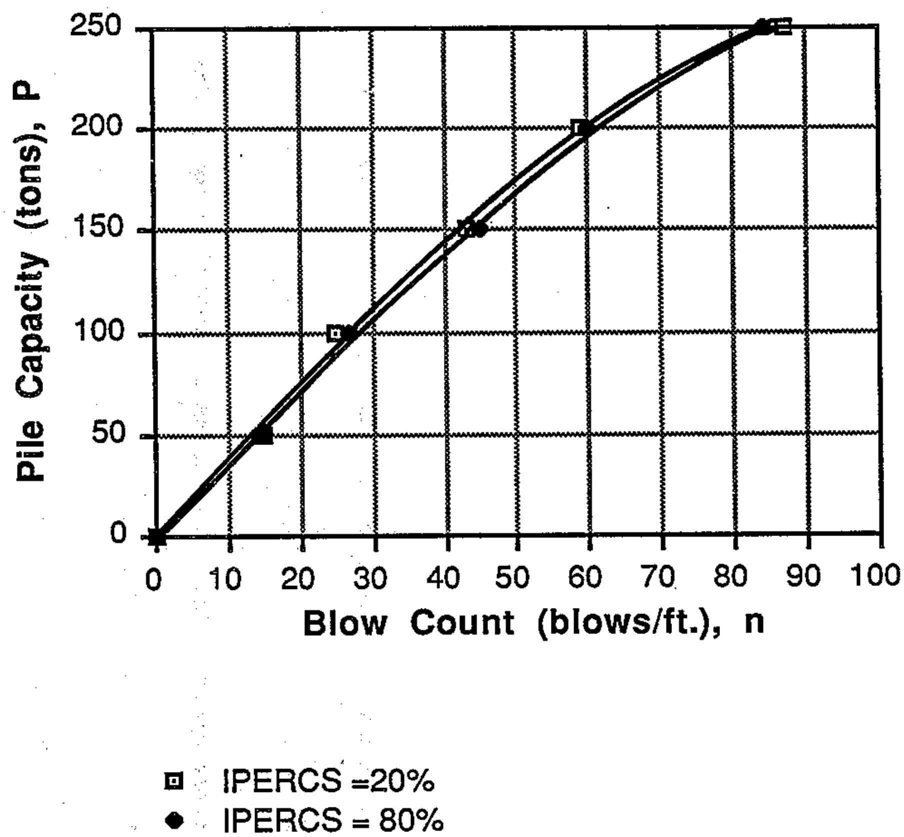


Fig. 17 - EFFECT OF IPERCS ON THE P-n CHART

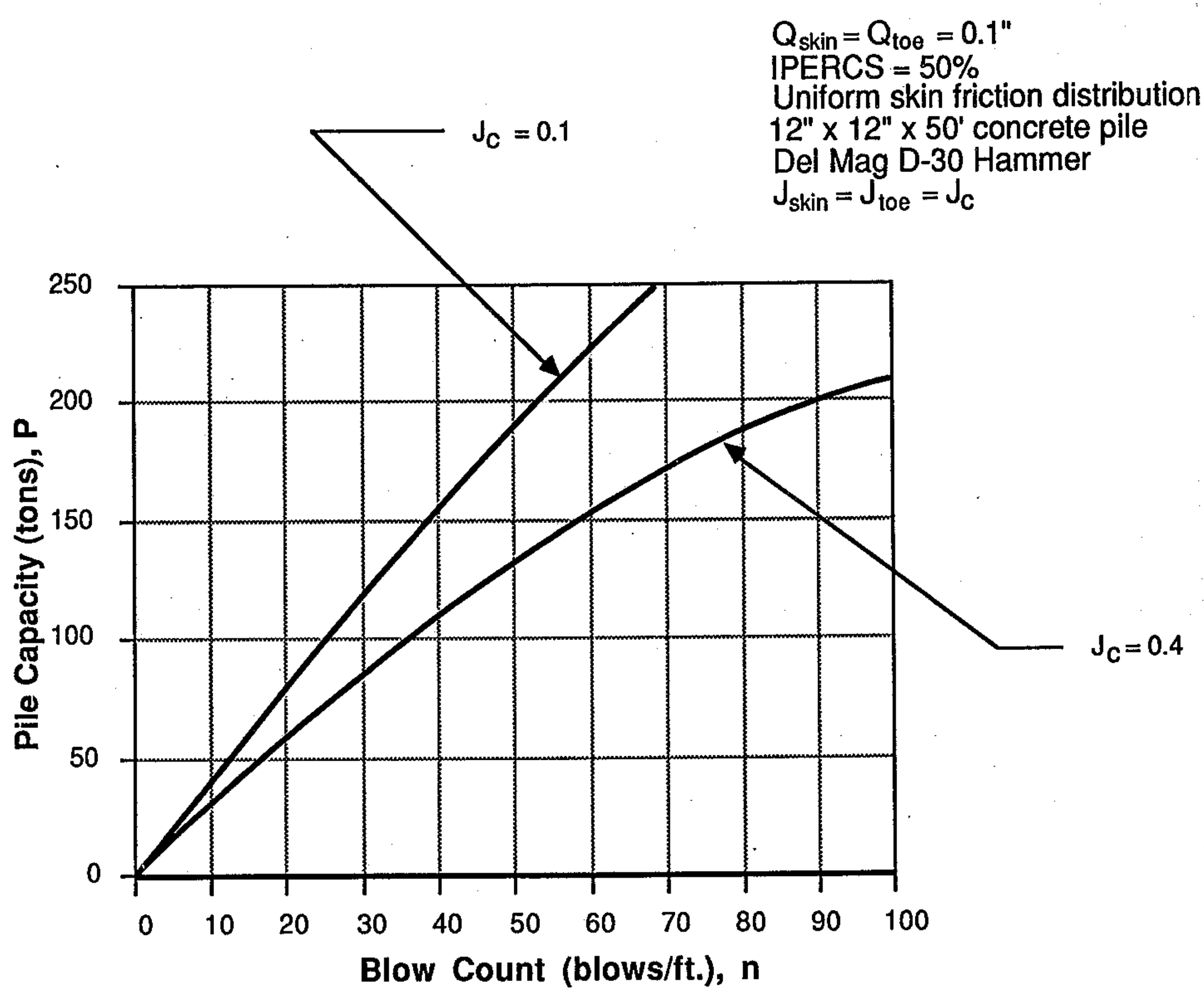
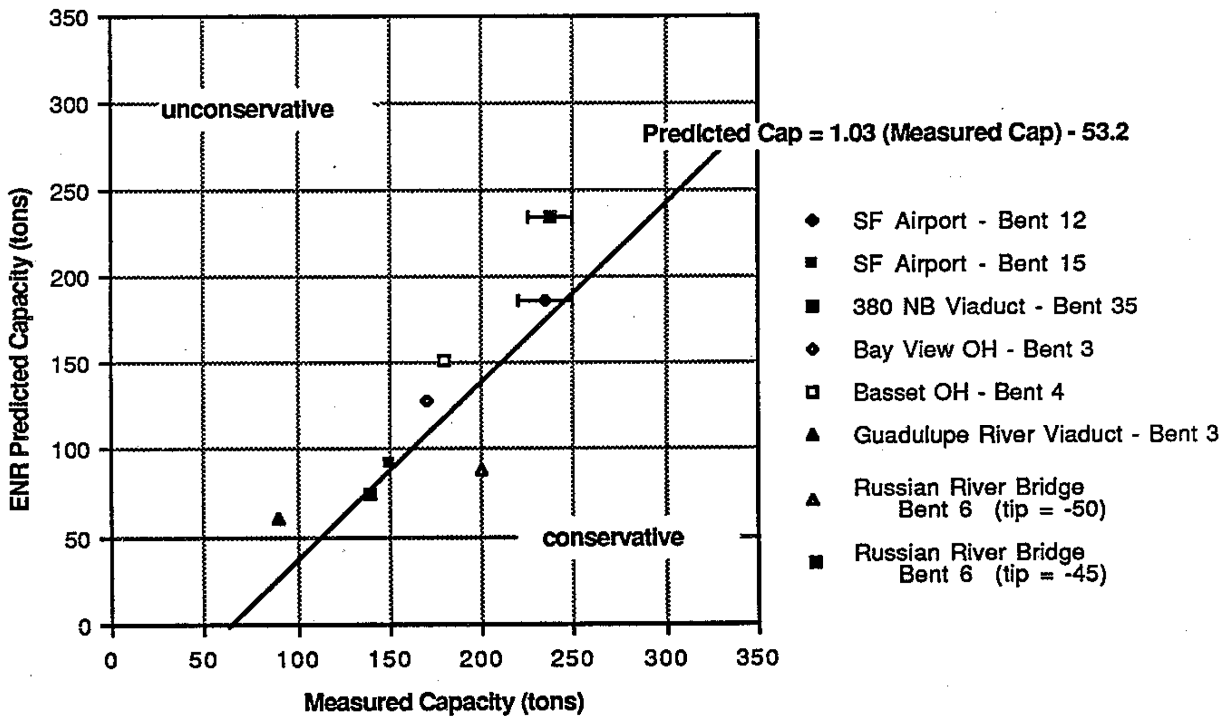


Fig. 18 - EFFECT OF CASE DAMPING CONSTANT ON THE P-n CHART

previous chapters, the stress wave generated while restriking a pile was measured using the PDA. The stress wave was then analyzed in order to determine the soil resistance mobilized under the hammer impact. A portion of the mobilized resistance is due to static soil resistance and the rest is due to dynamic soil resistance. The static soil resistance was then measured by a static load test. The measured static soil resistance was subtracted from the total soil resistance in order to determine the dynamic resistance mobilized under the hammer. With the dynamic soil resistance clearly established, the PDA damping constant (J_c) was determined. The PDA damping constant could then be used as input for the WEAP program.

The PDA assumes all damping to occur at the pile tip. WEAP assumes damping to occur at the pile tip and at the pile sides. For input into the WEAP program, JTOE can be taken as being equal to the PDA damping constant. Furthermore, it was found that a reasonable prediction of the P-n function could be achieved by using a Jskin equal to Jtoe in a WEAP analysis.

Fig. 19 shows static pile capacities predicted from WEAP generated P-n functions and the blow count measured during pile driving. WEAP input parameters determined by the methods outlined above were used to create the P-n functions. These predicted static capacities are plotted against the measured static pile capacities for piles tested to or almost to failure. Also in Fig. 19 is the ENR-predicted pile capacity versus the measured static pile capacity. The ENR-predicted capacity was also determined by using the blow count measured during pile driving. Best fit lines are included in both plots. The best fit for the WEAP-predicted data does not include the two data points shown for which failure was not reached (the two points showing ranges of measured capacity) because they were predicted to be out of the range of capacity normally encountered in highway construction. The best fit for the ENR-predicted data does include these data points because they were predicted to be within or near normal capacity. If either prediction scheme were perfect, all data points would plot directly on a 45 degree line. It can be seen from Fig. 19 that the WEAP-predicted capacities fall closer to a perfect 45 degree line than the ENR predicted capacities which are generally shifted to the conservative corner of the plot.



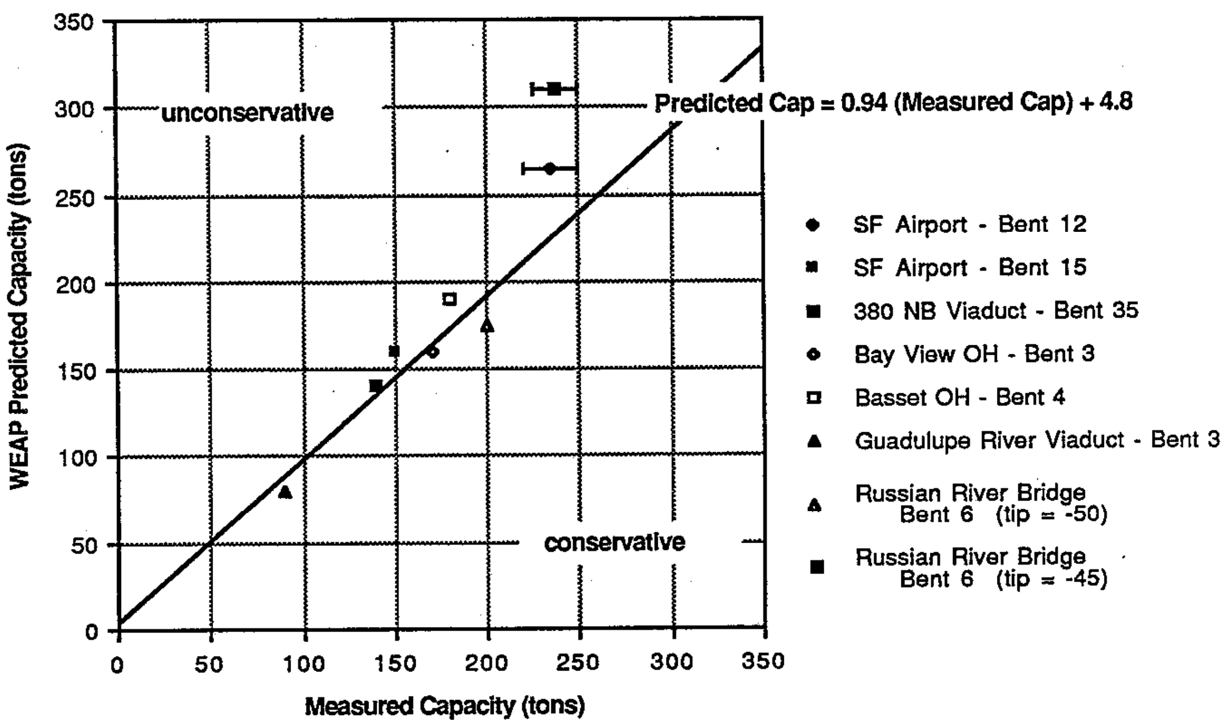


Fig. 19 - COMPARISON OF ENR & WEAP PREDICTED CAPACITY VS MEASURED STATIC PILE CAPACITY

C. Discussion

Installing a driven pile requires that the pile undergo displacements that would be characterized as failures when the pile is in service. Therefore, the very act of installing a driven pile presents the foundation engineer with an unique opportunity to assess its capacity. In order to do this however, the velocity-dependent forces generated while driving must be accounted for. The WEAP program offers a method of modeling the static, as well as the velocity-dependent, forces that occur when a pile is driven. This allows a means to more accurately predict pile capacity from information gathered during the installation process if the piling model is accurately described to the WEAP program.

As a means to this end, this study considered the procedures used in the WEAP program, and derived a method of describing the key input parameters to the program. The following is a summary of these parameters.

Hammer efficiencies typically range between 65% and 95% with 75% efficiency being the most common. If a WEAP analysis is performed for a hammer efficiency that is not near the efficiency of the hammer used to drive the pile, then inaccuracies in the analysis result. There is no way that the hammer efficiency can be known prior to pile driving. Therefore, a conservative approach should be used; an efficiency on the low side of this range should be used when finding the P-n function for a piling system and an efficiency on the high side of this range should be used when checking for pile overstress.

An accurate description of the piling equipment used by the contractor is also an important part of the WEAP input. In order to avoid miscommunication between the contractor and the engineer performing the analysis, a form similar to the one shown in Fig. 16 should be completed by the contractor.

It was found that the WEAP generated P-n chart was not sensitive to the percentage of pile capacity that is developed by skin friction (the input variable IPERCS). Also, work by others (2,3) indicates that static soil-pile interaction can be described by using skin and toe quakes equal to a tenth of an inch.

It was found that a reasonably accurate prediction of static pile capacity can be made by using, as input for WEAP, a J_{toe} and a J_{skin} equal to the J_c value defined in Fig. 10. This method describes dynamic soil-pile interaction to a sufficient degree such that a meaningful improvement in pile capacity prediction can be achieved relative to the capacity prediction made by the ENR formula.

SUMMARY AND CONCLUSION

Results of the static load tests and dynamic measurements have provided data regarding soil damping values that can be correlated with SPT values. The developed correlation involved mainly cohesionless and intermediate cohesionless soils. In most deep foundation construction, the pile tip will be founded in a sandy soil layer. Thus, available data for soil damping values correlated with unconfined compression test values were not obtained.

The measurements and tests also provided information on maximum hammer energy transferred to pile, hammer efficiency, and soil setup surrounding the pile shaft with respect to time. The following results found in this study and conclusions are:

1. The soil damping constant, J_c, can be determined by using the calculated restrike RMAX and measured static load bearing capacity, provided a pile failure criterion is defined. For piles with a small cross section, the defined criterion can be a limited settlement or vertical displacement when the pile is subjected to a vertical static load. The limited settlement can also be a plunging failure mode. For piles with a very large cross section, the plunging failure mode may not apply since the limited hammer or special designed drop weights may be inadequate to cause a sufficiently large permanent toe displacement or set. In this report, four static load tests for small piles resulted in plunging failure.

2. Soil setup will affect the determined J_c value. In most cases, a delay period of about 18 hours appears to be satisfactory for a restrike. However, special areas or soil conditions can be encountered where a longer delay period is necessary to adequately evaluate the soil setup. An example is Bassett O.H. Bent 4, San Jose, in stiff clay, as previously discussed in this report. The wave traces will change with a different time period restrike. This change can result in an increase of skin friction along the pile shaft.

Soil setup pile dynamic measurements at Patton O.C., Alameda, showed that the majority of soil setup may have occurred in 3 days. The 14x14 inch prestressed concrete pile was driven into soft to stiff clay and was 104 feet in length. The soil setup ratio after 18 hours was 2.7 and after 3 days was 4.0.

A steel pile driven into river pea gravel for the Gianelli Bridge, near Chico, revealed no soil setup after about 18 hours from a dynamically monitored restrike.

3. Predrilling will destroy the skin friction along the pile shaft during initial pile driving as shown by examining the force and velocity traces for the S.F. Airport Bents 21 and 35. However, a restrike showed that the soft clay tends to redistribute itself and develop skin friction along the pile shaft after 18 hours.

   In general, skin friction was observed to act on most of the embedded length of pile during restrike from examining the force and velocity traces.

4. Pile restrike calculations revealed that a significant increase in RMAX or RSU occurred in the 18 hour waiting period. The absolute RMAX or RSU can be much larger several months later as was found in test at NW21, San Mateo.

5. For a pile with very long penetration length, the velocity trace, due to skin friction, can become negative before 2 Lg/c. RSU is the bearing capacity computation applicable for this case and should be examined carefully since this estimated value may govern over RMAX.

6. The damping constant, J_c, is not only dependent on the soil type but also on the pile length. Pile penetration lengths less than 50 ft. were found to have smaller J_c values than for pile penetration lengths greater than 60 ft. This conclusion appears to be logical since the damping force is proportional to the pile tip velocity which in turn is a function of pile length.

7. A straight-line fit between J_c and SPT values in Fig. 10 offers a method for determining J_c for use in WEAP and PDA. In this case, the straight-line fit is based on a soil setup delay period of about 18 hours and an average soil setup ratio of about 2. The long setup delay period ratio appears to be about 3 to 4. Thus, the J_c values in this report will be on the conservative side.

   This study shows it is possible to establish a simple straight-line curve for J_c vs N for piles less than 50 ft in penetration length. For short piles, end bearing plays a more predominant role. For longer piles in which skin friction plays a more predominant role, it is difficult to quantify the J_c vs N into a single curve.

   Also, it appears that for SPT values greater that 90, pile lengths may have a small influence on the J_c value. Results from driving piles into very dense strata in this study suggests a minimum value for J_c is 0.10 for SPT values greater than 90. For SPT values greater than 70, a J_c value of 0.20 or less is suggested.

8. A study was made to consider the procedures used in the WEAP program and perform a parametric evaluation. Assuming all other input parameters to be reasonably correct, it was found that a reasonably accurate prediction of static pile capacity can be made using as input for WEAP, a J_{toe} and J_{skin} equal to the J_c value defined in Fig. 10. This assumed skin and toe quakes of 0.1 inch.

   One of the difficulties of the WEAP program is the evaluation of soil setup that might occur over a delay period. The established J_c values in Fig. 10 have accounted for a good part of the soil setup for use as J_{toe} and J_{skin}. This parametric study shows a possible improvement in the WEAP program pile capacity prediction. However, further evaluation is required, due to limited data.

RECOMMENDATIONS

This study considered the soil damping constant, J_c, in the wave equation procedure for calculating the ultimate bearing capacity of a pile. The J_c value is dependent on:

1. Soil type and density for the upper layers and below the pile tip.
2. The pile penetration length.
3. Soil setup.
4. Pile failure criterion.

A procedure has been proposed as shown in Fig. 10. Further study and testing are recommended due to limited data. For longer piles, 60 ft. or greater, the curve needs further adjustments and confirmation. Also, curves for pile lengths between 50 and 100 ft at 10 ft increments should be developed. This would offer increase accuracy of the damping constant with respect to pile penetration length.

The development of J_c curves can be used for the WEAP program. It was shown from a parametric study that reasonable answers can be obtained from J_c values obtained from the PDA and static load test results. Further study is recommended.

IMPLEMENTATION

The findings of this research study have been implemented on projects where pile driving construction monitoring is required. The J_c values, Fig. 10, are utilized in the ultimate bearing capacity calculations. Static load tests required in contracts have been eliminated in certain situations which have resulted in cost savings to the State.

The findings from this research project have also provided a better feel on how the WEAP program should be utilized. In other words, how the soil damping constants should be handled for the pile shaft and tip.

REFERENCES

1. Hough, B.K., "Basic Soils Engineering", Ronald Press Company N.Y., 1957 pg. 339.

2. Goble, G. G. and Rausche, F., "Wave Equation Analyses of Pile Driving, WEAP Program", Vol. 1 - Background, Vol. 2 - User's Manual, Vol. 3 - Program Documentation, U.S. Department of Transportation. FHWA Vol. 76-14.1 76-14.3 (updated 1981).

3. Pile Dynamics Inc. "Model GB, Pile Driving Analyzer Manual, January 1984.

4. Pile Dynamics Inc. "PDA Training Schedule", January 1984.

5. Timoshenko, S. and Goodier, J.N., "Theory of Elasticity", McGraw-Hill Book Co., 2nd Edition, 1951.

6. Koten, H. Van and Middendorp P., "Testing of Foundation Piles", Heron Vol. 26, 1981, No. 4.

7. Peck, R.B., Hanson, W.E., and Thornburn, T.H., "Foundation Engineering", John Wiley & Sons, Inc., 2nd Edition, Pg. 215, 1974.

8. Vesic, A.S., "Design of Pile Foundations", National Cooperative Highway Research Program Synthesis of Highway Practice 42, Transportation Research Board National Research Council, Washington, D.C., 1977, Pg. 10.

9. Tomlinson, M.J., "Pile Design and Construction Practice", Garden City Press Limited, Letchworth, Hertfordshire, SG61JS, 1977, 1981, Pg. 119.

10. Schertmann, John H., "Use of SPT to Measure Dynamic Soil Properties - Yes, But .....!," Highway Focus, Vol. 12, No. 2, U.S. Department of Transportation, FHA, April 1980.

11. Gallet, A.J., "Use of the Wave Equation to Investigate Standard Penetration Test Field Measurements Master of Engineering Thesis", University of Florida, 1976.

12. Marcuson, W.F., and Bieganousky, Wayne A., "Laboratory Standard Penetration Tests on Fine Sands", JGED, ASCE, GT6, Pg. 565, June 1977.

13. Smith, E.A.L., "Pile Driving Analysis by the Wave Equation", J. SMD, ASCE, Vol. 86, Sm4, Pg. 35-61, August 1960.

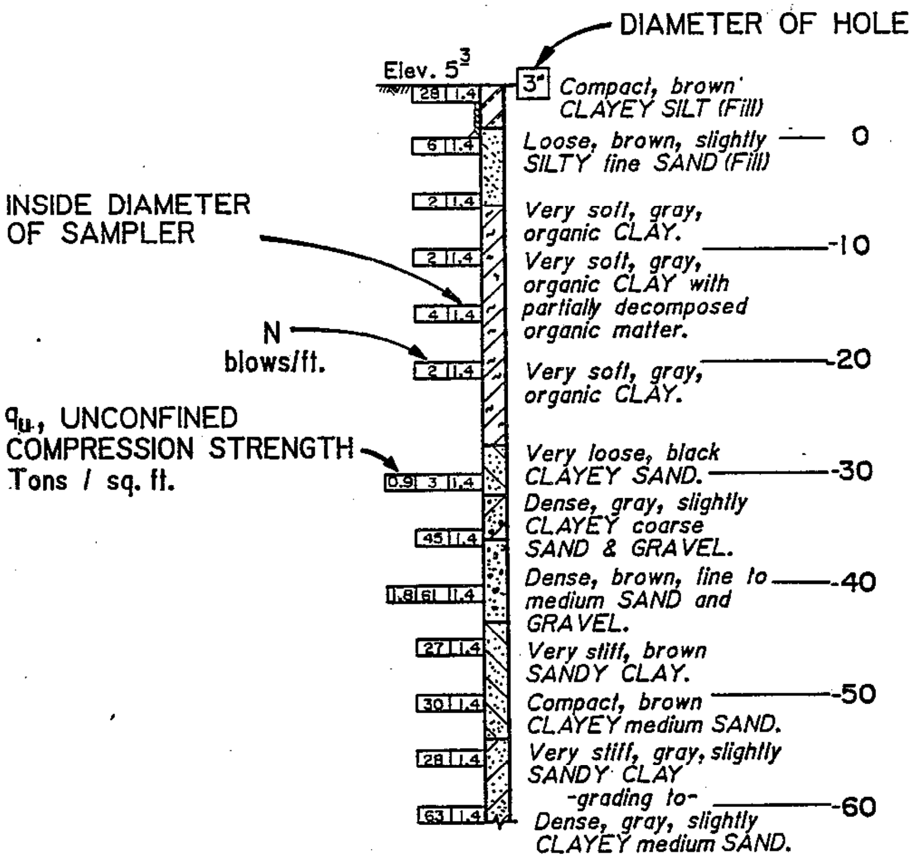
14. Lowery, L.L., Hirsch, T.J., and Samson Jr., C.H., "Pile Driving Analysis - Simulation of Hammers, Cushions, Piles, and Soil", Texas Transportation Institute, Research Report, No. 33-9, 1967.

15. Goble, G.G., Likins Jr., G., Rausche, F., "Bearing Capacity of Piles from Dynamic Measurement, Ohio - DOT, 05-75, U.S. Department of Transportation, FHA, 1975, Pg. 27.

16. Litkouhi, S., Poskitt, T.J., "Damping Constants for Pile Driveability Calculations", Geotechnique, Vol. 30, Number 1, March 1980.

17. Authier, J. and Fellenius, B.H., "Wave Equation Analysis and Dynamic Monitoring of Pile Driving", Civil Engineering for Practicing and Design Engineers, Pergamon Press. Ltd., Vol. 2, Pg. 387-407, 1983.

APPENDIX



LEGEND OF BORING

ENR.............ENGINEERING NEWS RECORD FORMULA

$$ENR = \frac{2(W)\ (STROKE)}{S + 0.1}$$

WHERE $S = 12/b$ W = Weight of hammer
b = blows/ft.

TR................ENERGY TRANSFER RATIO (%)

DEFINITIONS

RTL, RMX (RMAX), RSU, RMN, RAU are defined on Page 19 (kips)

RS1, RS2, Static resistances for RSP at peaks 1 and 2 (kips)

FMX (FMAX), FT1, FT2, forces at peaks 1 and 2 (kips)

VMX (V_{MAX}), VT1, VT2, Velocities at peaks 1 and 2 (ft/sec) Readings divided by 10

DMX, DT1, DT2, Downward displacement of pile at transducer location per blow of hammer at peaks 1 and 2 (inches). Readings divided by 100

BPM---Defined on page 20

TMX---Defined on page 20 (msec) Readings divided by 10

TMN---Time 2L/C of minimum capacity (msec) Readings divided by 10

EMX (EMAX) defined on page 19. (kip-ft) Readings divided by 10

WUP (Wu), WDN (Wd) defined on page 19 (kips)

$2L/c = 2L_g/c$---time peak 1 to peak 2 (msec) Readings divided by 10

EA/c---Pile impedance $(\frac{\text{kips-sec}}{\text{ft}})$ Readings divided by 10

J_c---Case damping constant, readings divided by 100

Δ---Time delay method (msec)

SFT---Skin friction total (kips)

CTN---Maximum computed tension force (kips)

DEFINITIONS

The following data for bearing pile are listed:

12 x 12 Prestressed Concrete Pile Specified Tip Elev. +20

L (ft)	L_g (ft)	BPM	Stroke (ft)	TR (%)	c ft/sec	Hammer
Initial						
82	79.25	50	5.3	34	11,917	FEC 3000
Restrike						
82	79.25	48	6.9	38	11,917	

Anchor pile L was 87 ft E = 4594 ksi

Initial Dynamic Test		119 Tons	J_c = 0.55
Restrike Dynamic Test		275 Tons (RMAX) 322 Tons (RSU)	J_c = 0.55
Initial ENR	36 b/ft	81 Tons	
Restrike ENR	17 b/inch	N/A	
Static Load Test		200 Tons	0.38" Settlement
Anchor Pile Pullout		100 Tons	0.48" Uplift

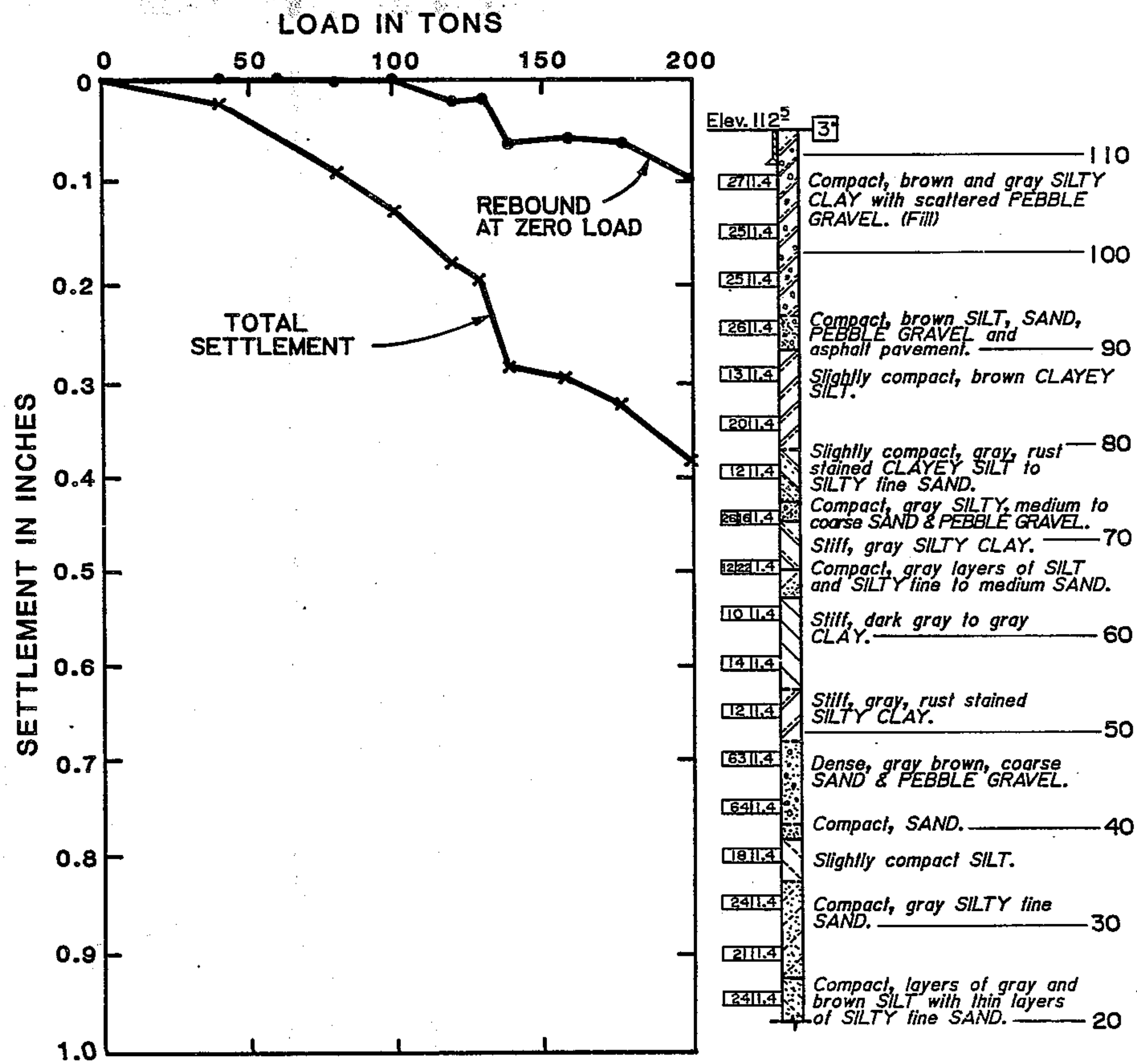
Anchor pile began creeping up at 90 tons

Restrike Period - 18 hours

Static Load Period - 3 days after restrike

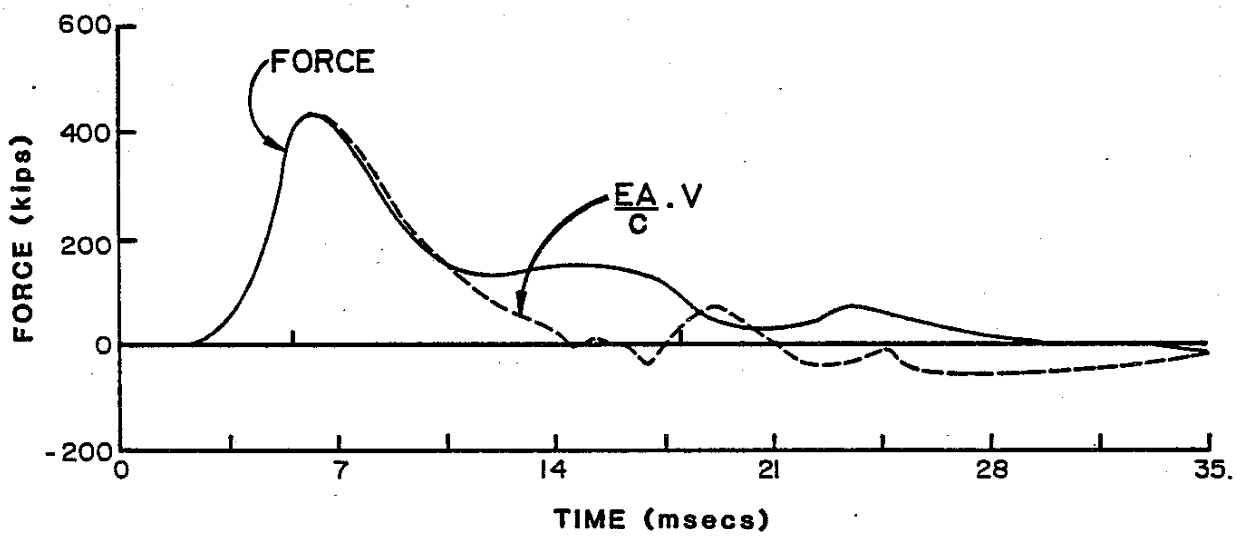
Penetration L - 78.5 ft

BEARING PILE TEST DATA
Abutment 3, Park Avenue U.C., SAN JOSE

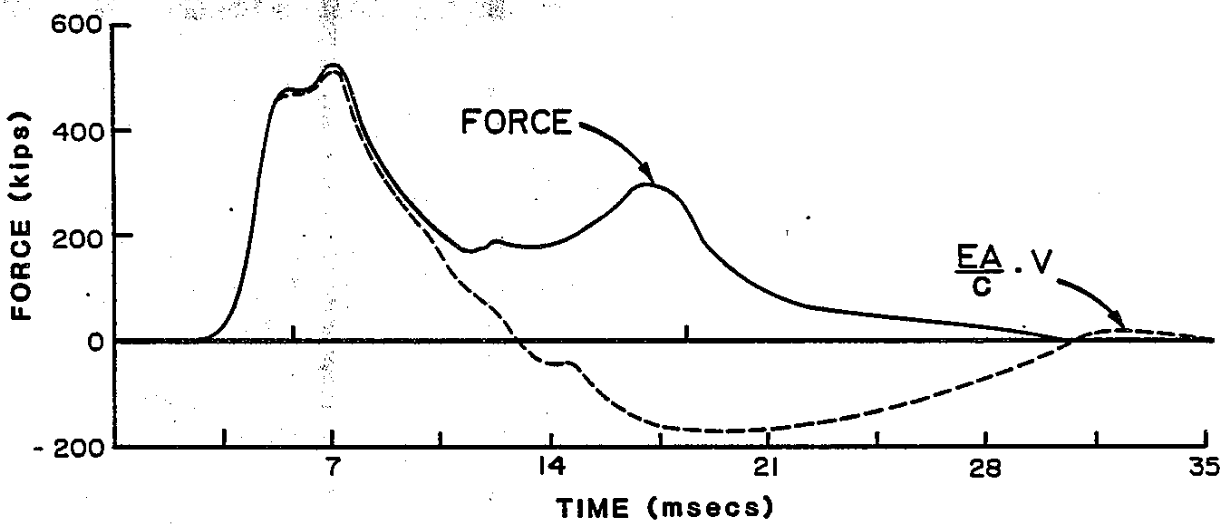


STATIC LOAD TEST RESULTS AND LOG OF TEST BORING

Abutment 3, Park Avenue U.C., SAN JOSE



INITIAL WAVE TRACES
Abutment 3, Park Avenue U.C., SAN JOSE



RESTRIKE WAVE TRACES
Abutment 3, Park Avenue U.C., SAN JOSE

The following data for bearing pile are listed:

12 x 12 Prestressed Concrete Pile   Specified Pile Tip +8

L (ft)	L_g (ft)	BPM	Stroke (ft)	TR (%)	c ft/sec	Hammer
Initial						
63.3	60.8	60	4.0	44	12,040	FEC 3000
Restrike						
63.3	60.8	48	6.0	25	12,040	

Anchor pile L was 78.3 ft   E = 4689 ksi

Initial Dynamic Test	57 Tons	J_c = 0.50
Restrike Dynamic Test	108 Tons (RMAX) 137 Tons (RMAX)	J_c = 0.50 J_c = 0.35
Initial ENR 14 b/ft	28 Tons	
Restrike ENR 37 b/6"	152 Tons	
Static Load Test	140 Tons	0.35" Settlement
Anchor Pile Pullout	70 Tons	0.08" Uplift

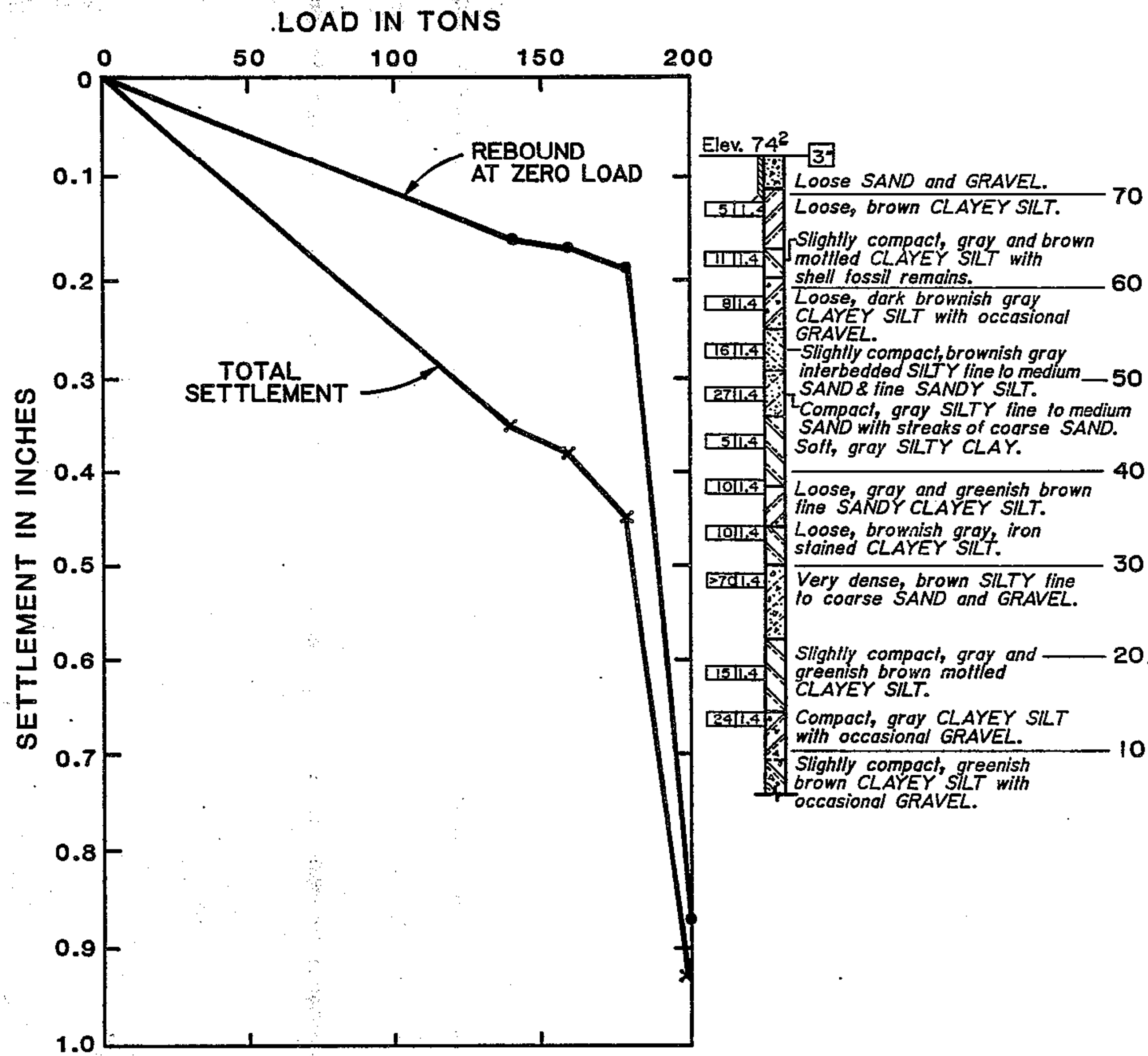
Restrike Period - 18 hours

Static Load Period - 6 days after restrike

Penetration L - 60 ft

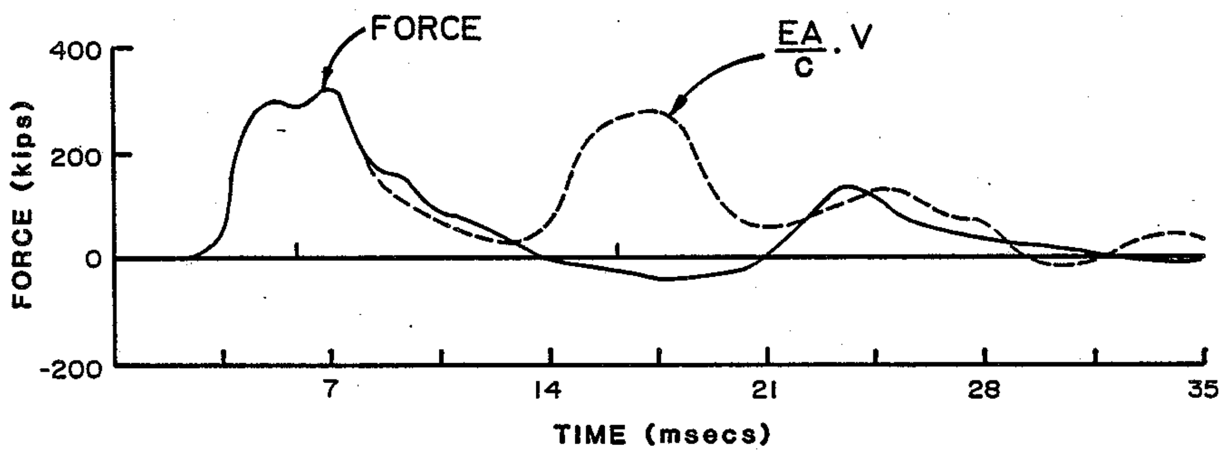
Bearing Pile failed by plunging at about 180 tons

BEARING PILE TEST DATA
Bent 4, Bassett OH, SAN JOSE

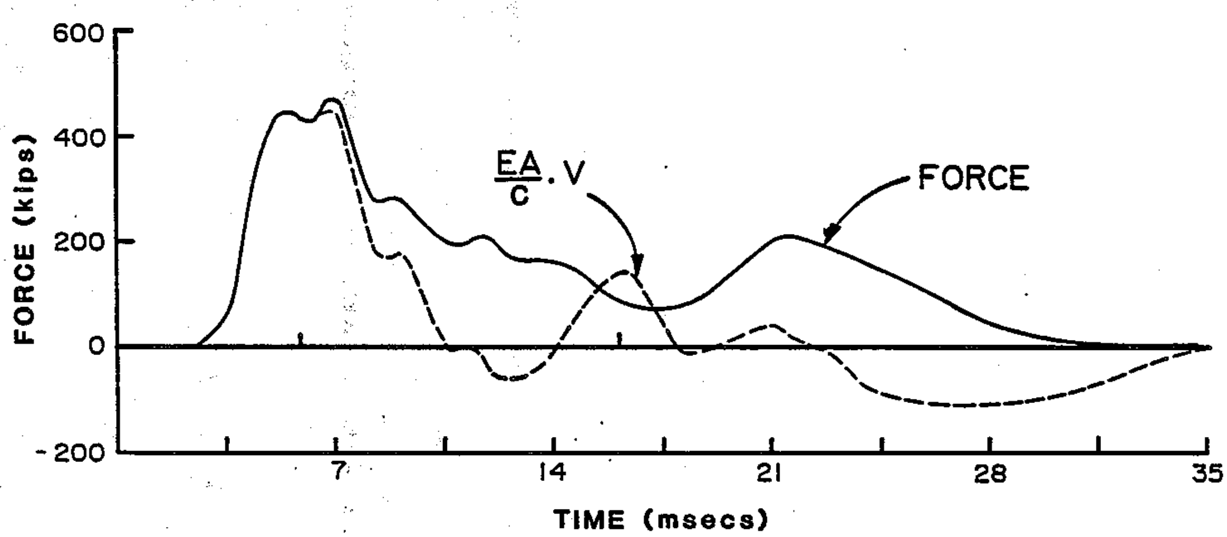


STATIC LOAD TEST RESULTS AND LOG OF TEST BORING

Bent 4, Bassett O.H., SAN JOSE



INITIAL WAVE TRACES
Bent 4 Bassett O.H., SAN JOSE



RESTRIKE WAVE TRACES
Bent 4, Bassett O.H., SAN JOSE

The following data for bearing pile are listed:

12 x 12 Prestressed Concrete Pile Specified Tip Elev. +42

L (ft)	L_g (ft)	BPM	Stroke (ft)	TR (%)	c ft/sec	Hammer
Initial						
42	39.5	49	5.7	35	11,970	Del Mag 30-23
Restrike						
42	39.5	45	6.8	38	11,970	

Anchor pile L was 42 ft

E = 4635 ksi.

Initial Dynamic Test	42 Tons	J_c = 0.30
Restrike Dynamic Test	87 Tons (RMAX)	J_c = 0.30
Initial ENR 7 b/ft	20 Tons	
Restrike ENR 26 b/ft	79 Tons	
Static Load Test	80 Tons	0.18" Settlement
Anchor Pile Pullout	45 Tons	0.07" Uplift

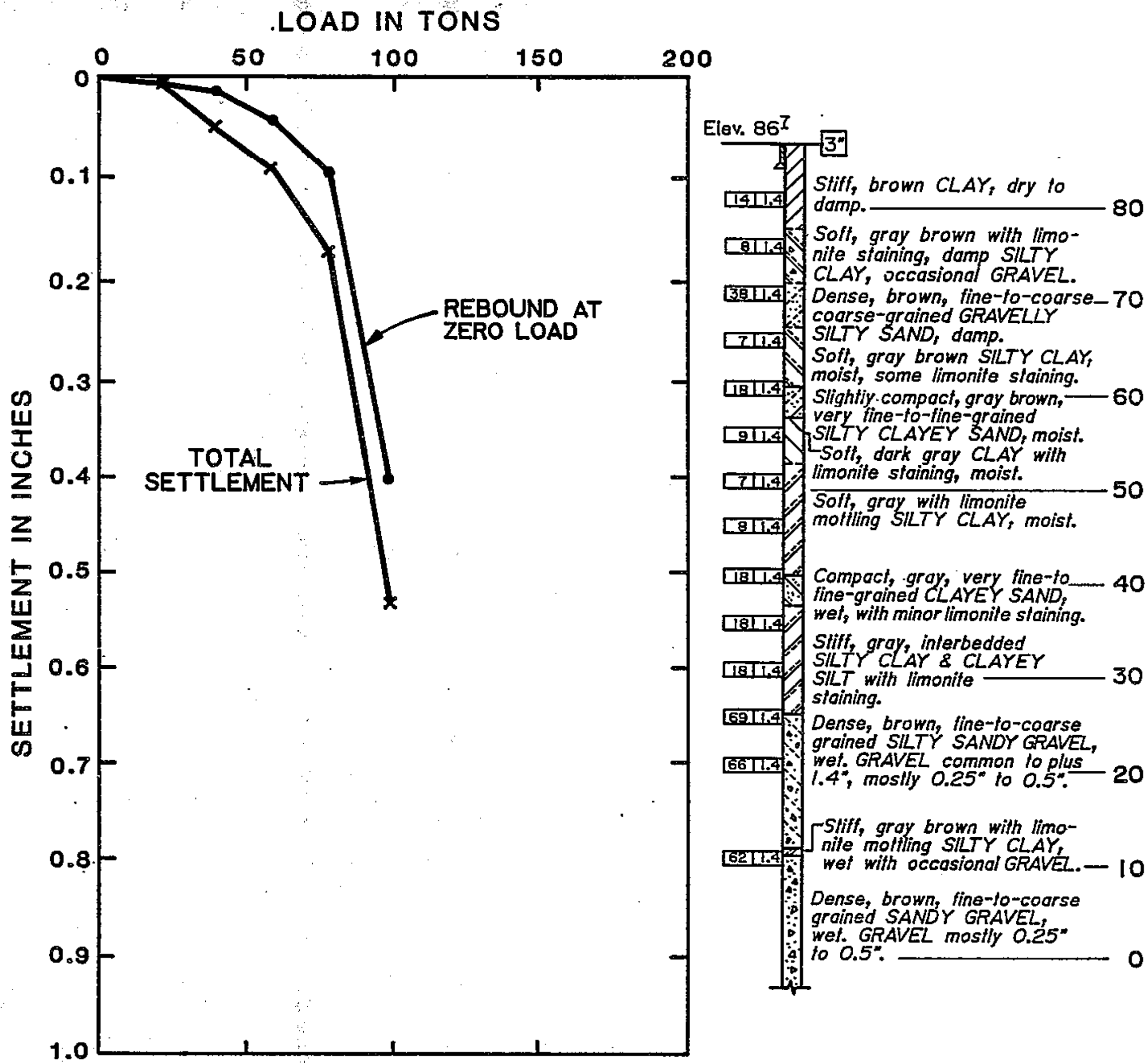
The bearing pile failed by plunging at 100 tons

Restrike Period - 18 hours

Static Load Period - 6 days after restrike

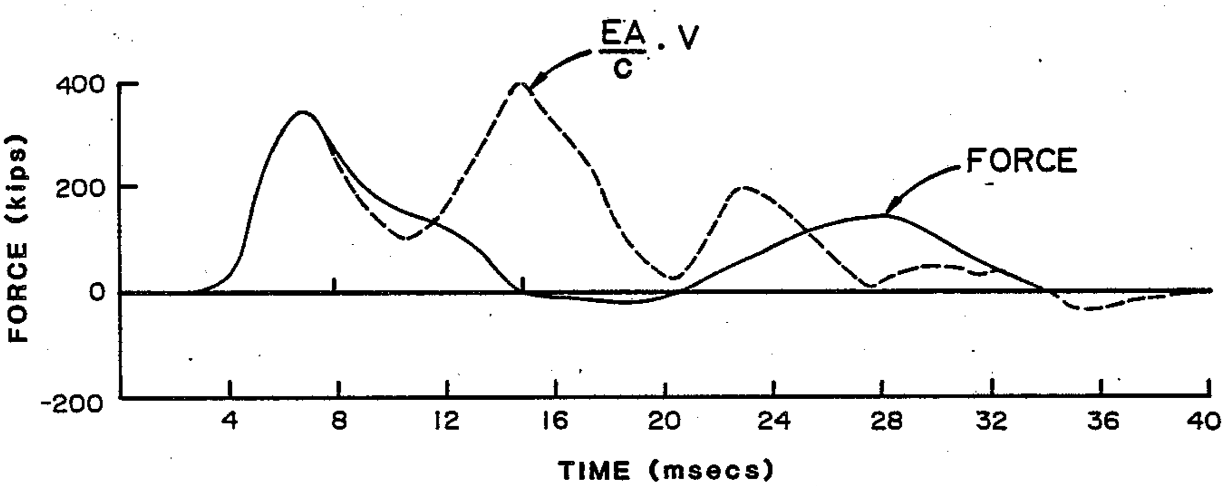
Penetration L - 36.2 ft

BEARING PILE TEST DATA
Bent 3, Guadalupe River Viaduct, SAN JOSE

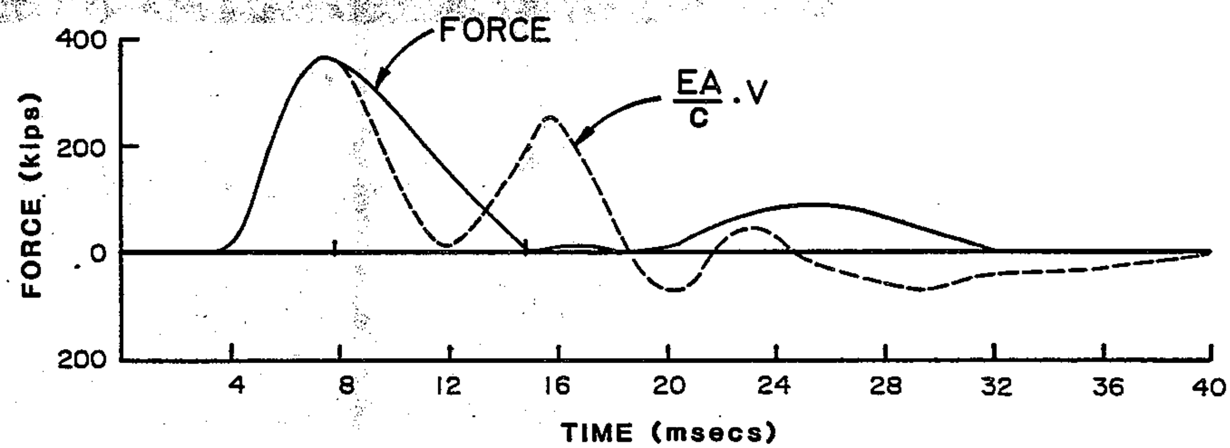


STATIC LOAD TEST RESULTS AND LOG OF TEST BORING

Bent 3, Guadalupe River Viaduct, SAN JOSE



INITIAL WAVE TRACES
Bent 3, Guadalupe River Viaduct, SAN JOSE



RESTRIKE WAVE TRACES

Bent 3, Guadalupe River Viaduct, SAN JOSE

The following data for bearing pile are listed:

12 x 12 Prestressed Concrete Pile   Specified Tip Elev. -72

L (ft)	L_g (ft)	BPM	Stroke (ft)	TR (%)	c ft/sec	Hammer
Initial						
72	70	47	6.2	36	12,800	KC 35
Restrike						
72	70	39	9.2	31	12,800	

Anchor pile L was 72 ft   E = 5477 ksi

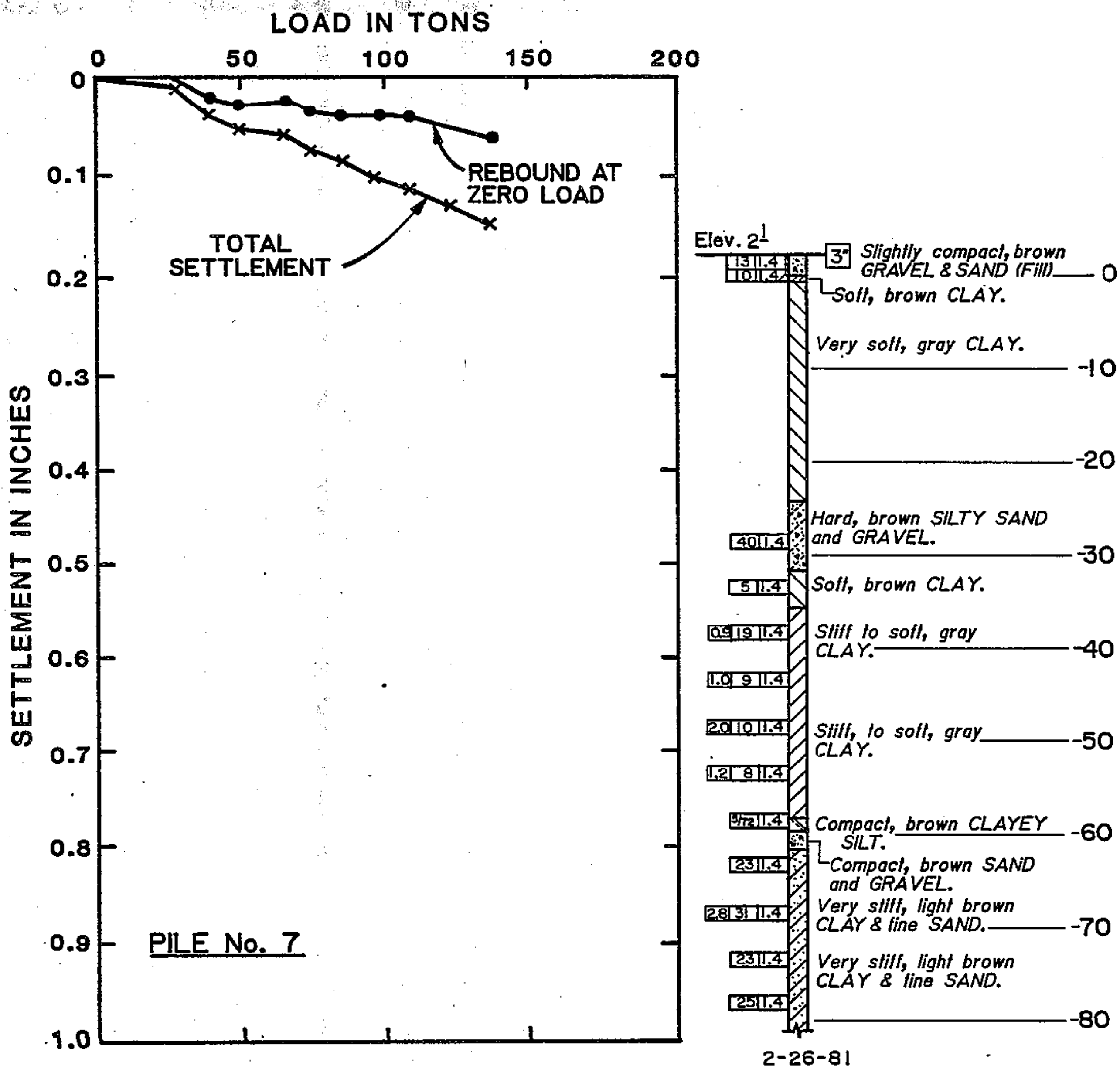
Initial Dynamic Test	69 Tons (ANCHOR PILE) J = 0.50	
Restrike Dynamic Test	245 Tons (RMAX) (ANCHOR PILE) J = 0.50	
Initial ENR 8 b/ft	30 Tons (1 HR RESTRIKE)	
Restrike ENR 44 b/ft	192 Tons	
Static Load Test	140 Tons (BEARING PILE)	0.16" Settlement Pile
Anchor Pile Pullout	70 Tons	N/A

Restrike Period - 60 days

Static Load Period - 7 days after initial drive

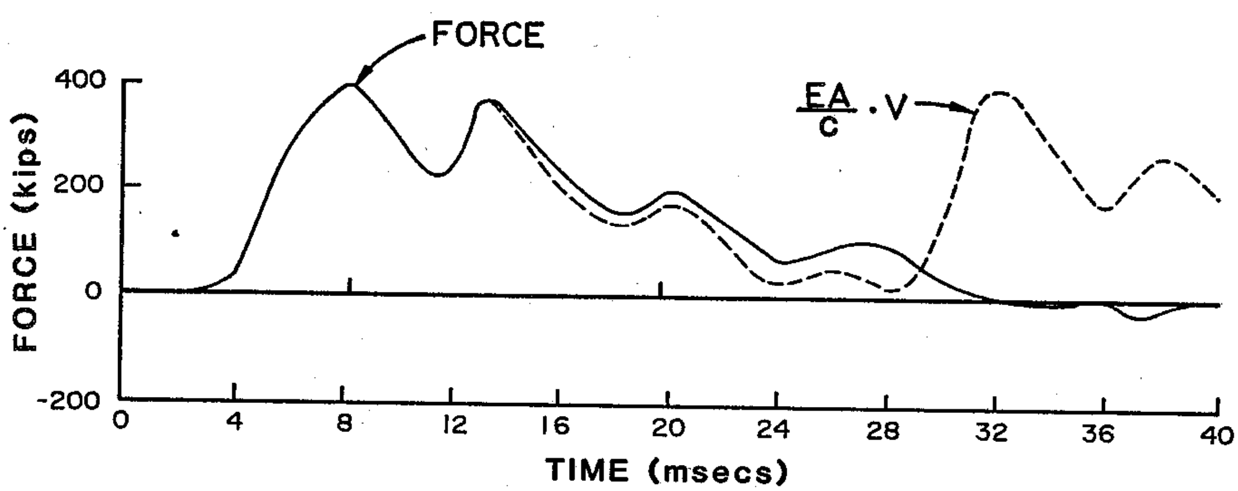
Penetration L - 69 ft

BEARING PILE TEST DATA
Bent 21, Northwest Connector, SAN MATEO

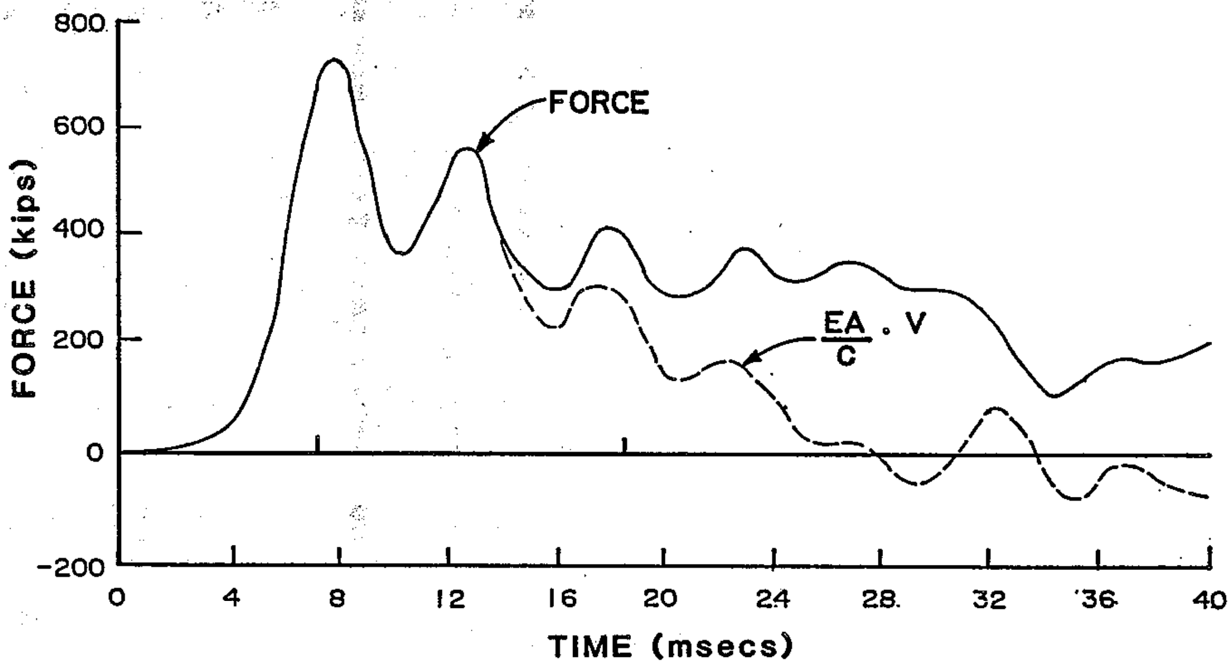


STATIC LOAD TEST RESULTS AND LOG OF TEST BORING

Bent 21, Northwest Connector, SAN MATEO



INITIAL WAVE TRACES (Pile No. 16)
Bent 21, Northwest Connector, SAN MATEO



RESTRIKE WAVE TRACES (Pile No. 16)
Bent 21, Northwest Connector, SAN MATEO

The following data for bearing pile are listed:

14HP89 Steel Piles Specified Tip -50

L (ft)	L_g (ft)	BPM	Stroke (ft)	TR (%)	c ft/sec	Hammer
Initial						
60	58	47	6.2	36	16,800	K25
Restrike						
None						

Anchor pile L was 60 ft E = 29,826 ksi

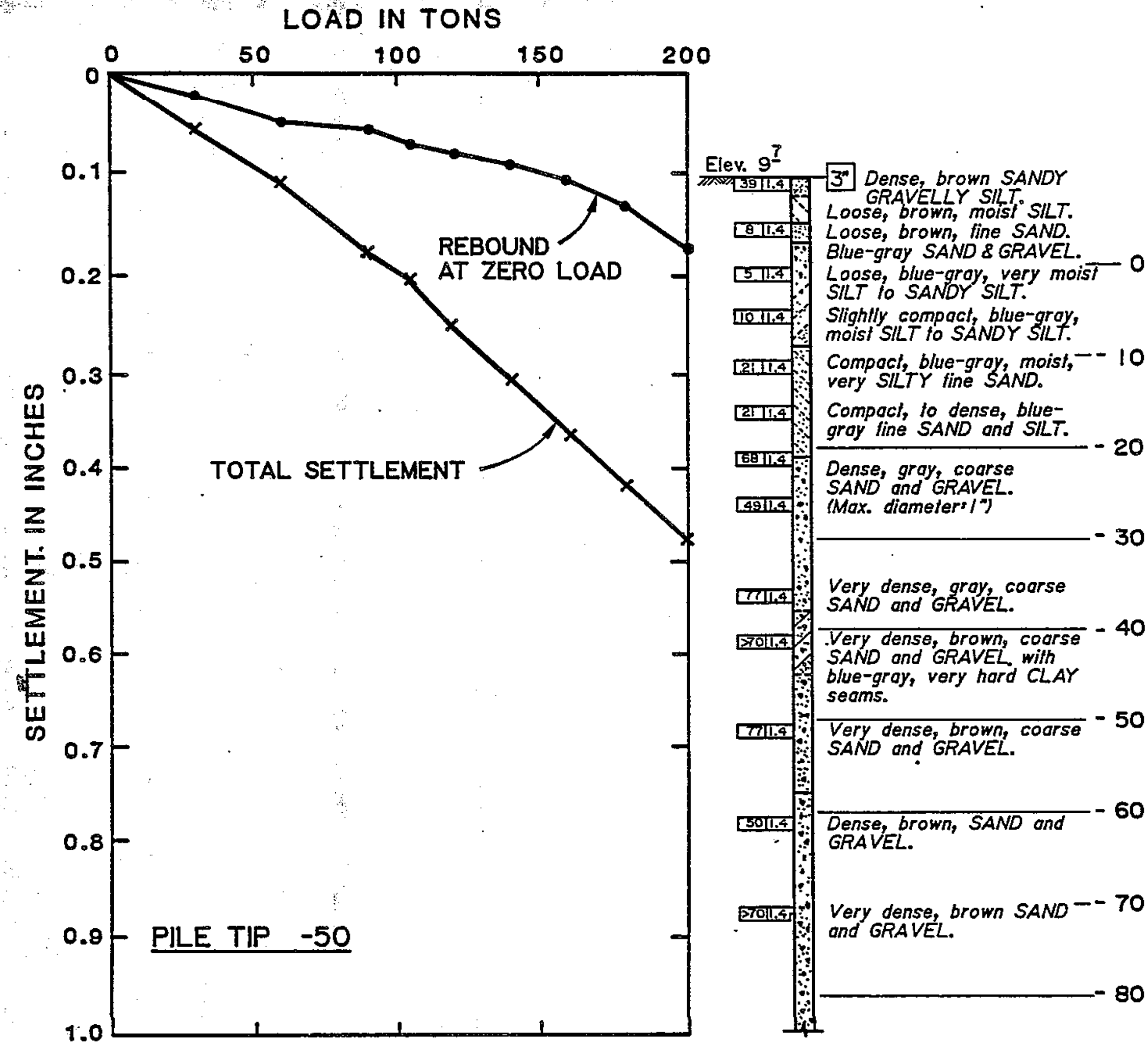
Initial Dynamic Test	190 Tons (RMAX) J_c = 0.12	
Restrike Dynamic Test	N/A	
Initial ENR 42 b/ft	88 Tons	
Restrike ENR	N/A	
Static Load Test	200 Tons	0.47" Settlement
Anchor Pile Pullout	100 Tons Start pullout @ 80T	0.67" Uplift 0.34" Uplift

Restrike Period - None

Static Load Period - 7 days after initial drive

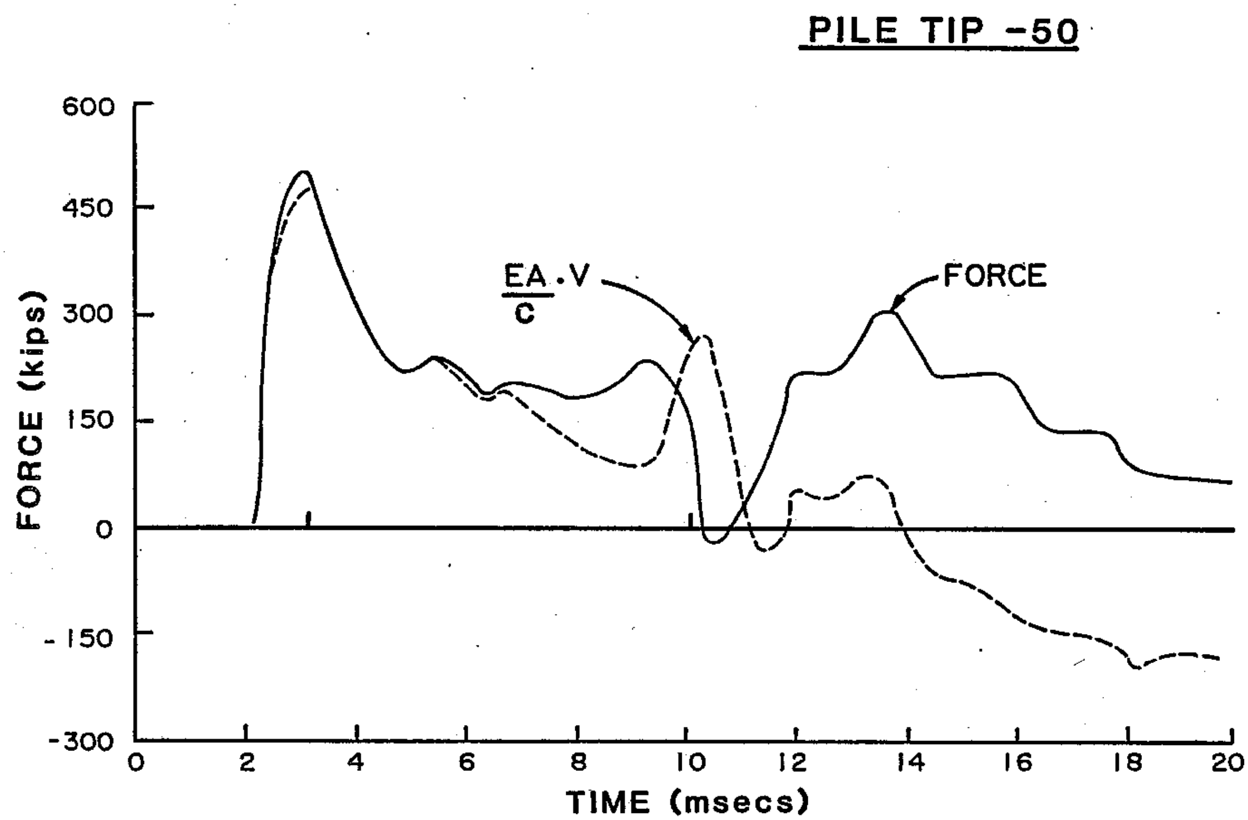
Penetration Length 41 ft

BEARING PILE TEST DATA
Bent 6, Russian River Bridge, JENNER



STATIC LOAD TEST RESULTS AND LOG OF TEST BORING

Bent 6, Russian River Bridge, JENNER



INITIAL WAVE TRACES
Bent 6, Russian River Bridge, JENNER

The following data for bearing pile are listed:

14HP89 Steel Piles   Specified Tip Elev. -45

L (ft)	L_g (ft)	BPM	Stroke (ft)	TR (%)	c ft/sec	Hammer
Initial						
60	58	49	6.5	34	16,800	K25
Restrike						
None						

Anchor pile L was 60 ft $E = 29{,}826$ ksi

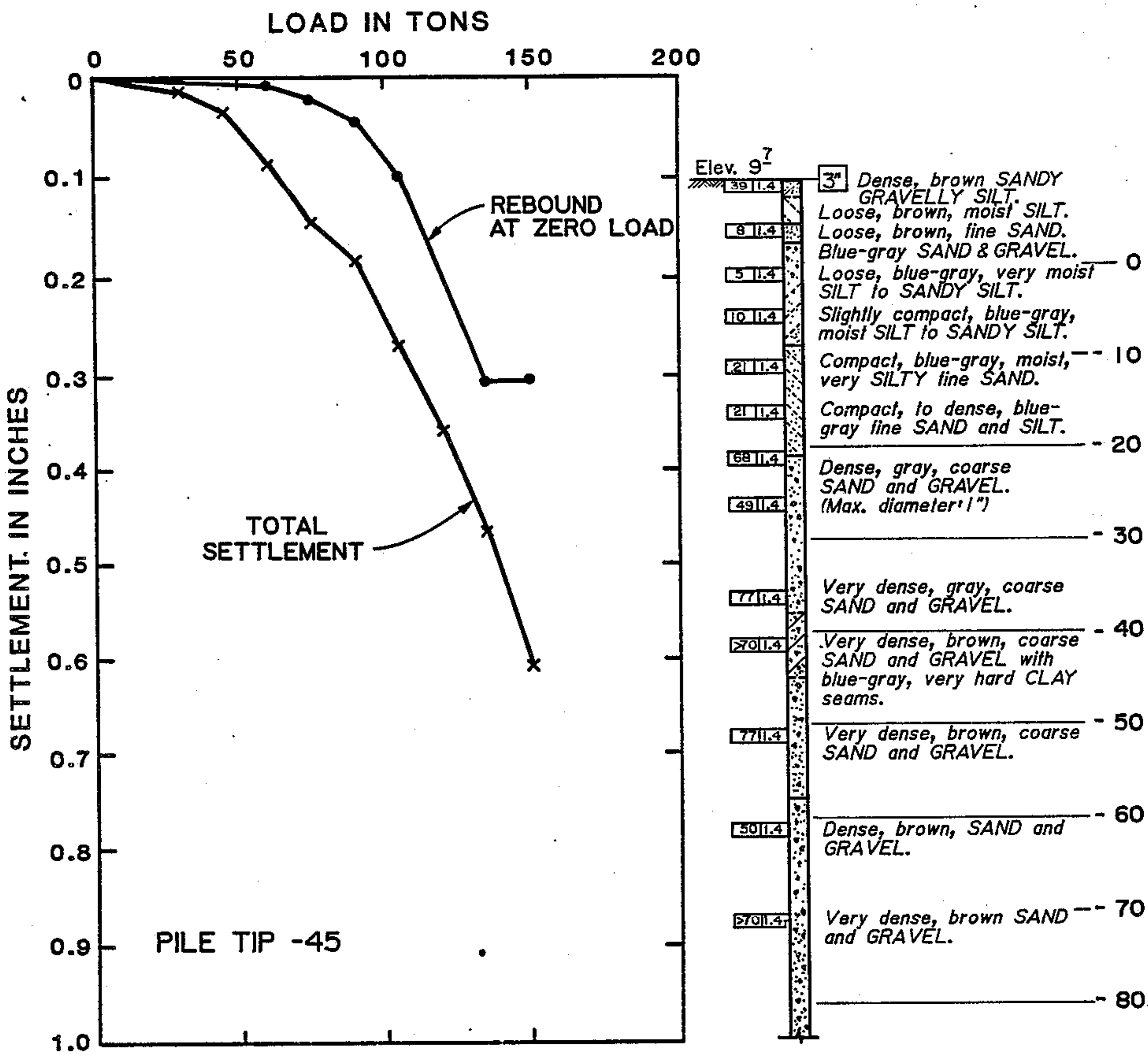
Initial Dynamic Test	155 Tons (RMAX) $J_c = 0.18$	
Restrike Dynamic Test	N/A	
Initial ENR 32 b/ft	75 Tons	
Restrike ENR	N/A	
Static Load Test	150 Tons	0.60" Settlement
Anchor Pile Pullout	75 Tons	0.75" Uplift
	Start pulling out at 60 tons 0.32" Uplift	

Restrike Period - None

Static Load Period - 10 days

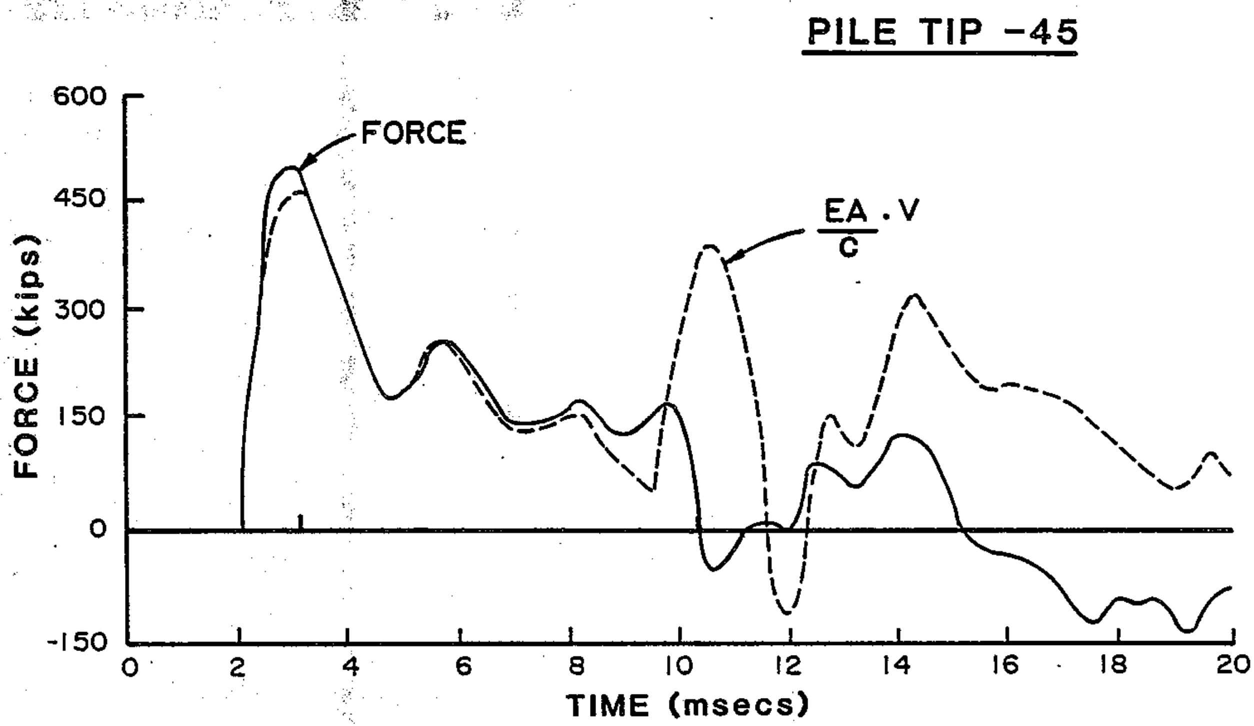
Penetration L - 36 ft

BEARING PILE TEST DATA
Bent 6, Russian River Bridge, JENNER



STATIC LOAD TEST RESULTS AND LOG OF TEST BORING

Bent 6, Russian River Bridge, JENNER



INITIAL WAVE TRACES
Bent 6, Russian River Bridge, JENNER

The following data for bearing pile are listed:

12 x 12 Prestressed Concrete Pile Specified pile tip Elev. +70

L (ft)	L_g (ft)	BPM	Stroke (ft)	TR (%)	c ft/sec	Hammer
Initial						
44.5	42.5	51	5.3	44	12,878	FEC 3000
Restrike						
None						

Anchor pile L was 44.5 ft E = 5444 ksi

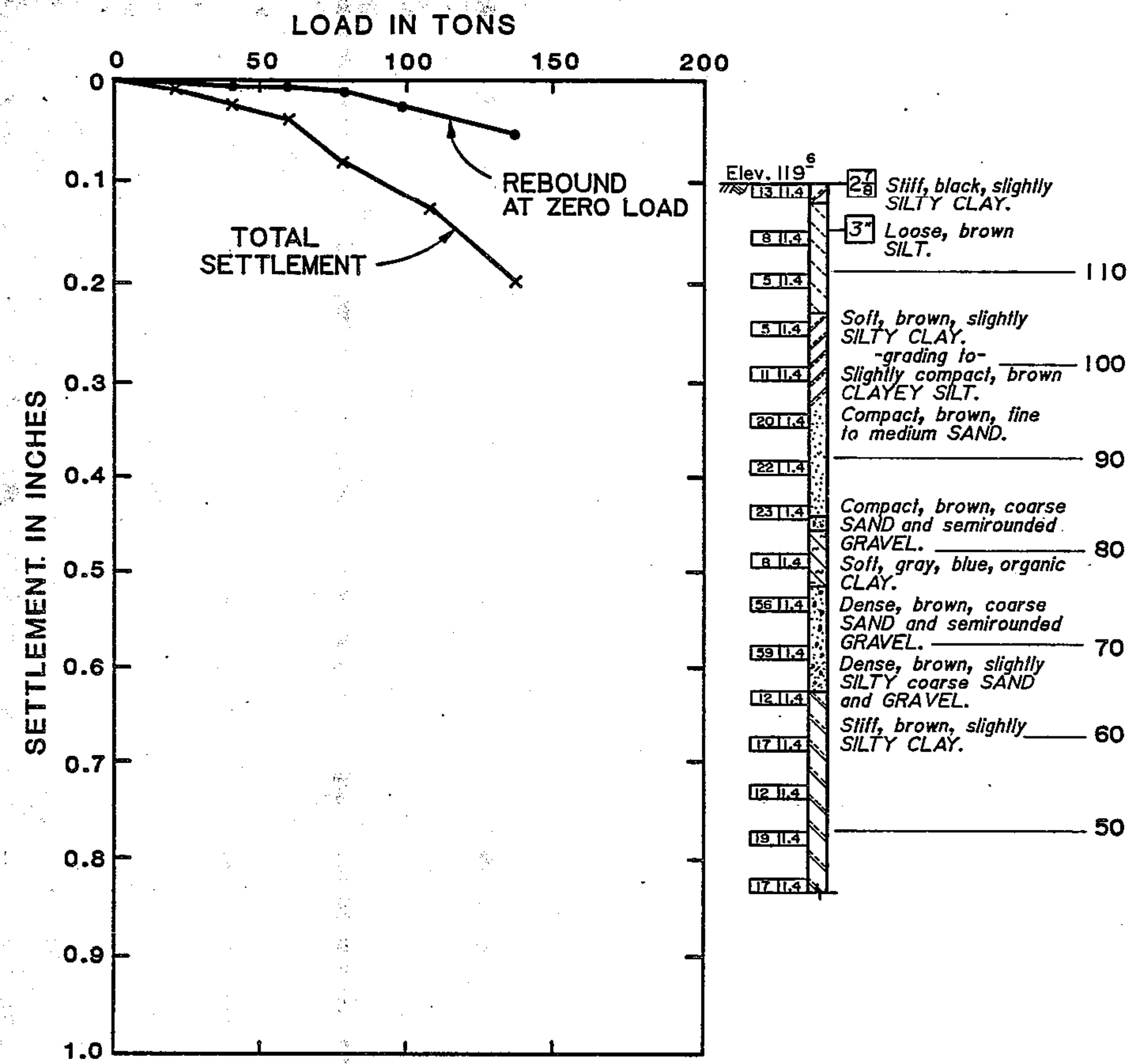
Initial Dynamic Test	163 Tons (RMAX) $J_c = 0.20$	
Restrike Dynamic Test	N/A	
Initial ENR 34 b/ft	78 Tons	
Restrike ENR	N/A	
Static Load Test	140 Tons	0.20" Settlement
Anchor Pile Pullout	70 Tons	0.12" Uplift

Restrike Period - None

Static Load Period - 6 days after initial drive

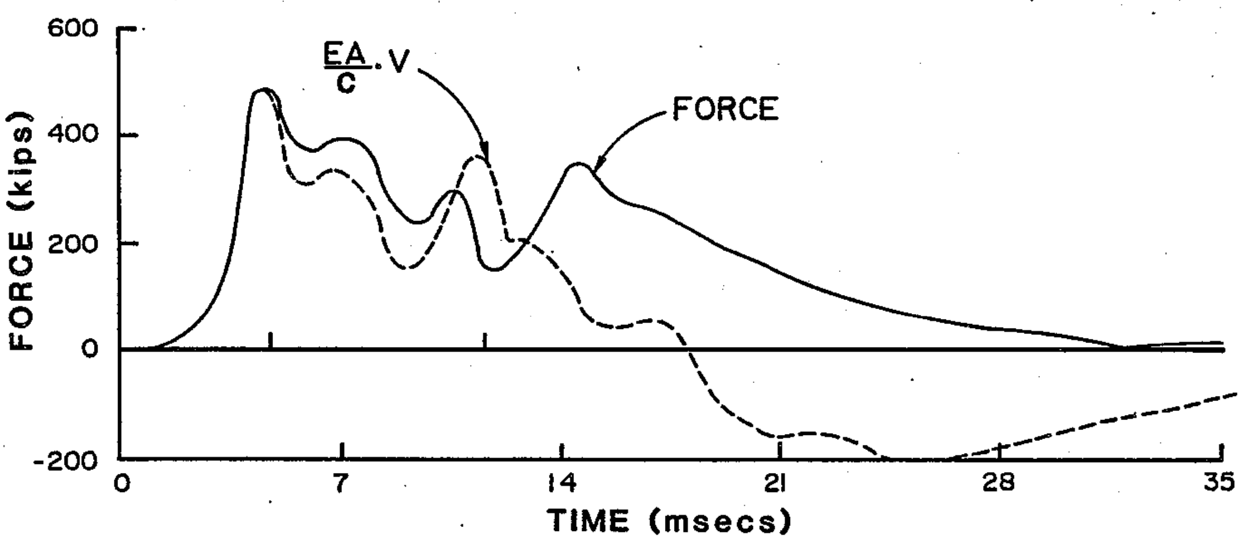
Penetration L - 41.5 ft

BEARING PILE TEST DATA
Bent 25 Newark Seal Slab, NEWARK



STATIC LOAD TEST RESULTS AND LOG OF TEST BORING

Bent 25, Newark Seal Slab, NEWARK



INITIAL WAVE TRACES
Bent 25-Newark Seal Slab, NEWARK

The following data for bearing pile are listed:

15" Octagonal Prestressed Concrete Pile Specified Pile Tip Elev. -38

L (ft)	L_g (ft)	BPM	Stroke (ft)	TR (%)	c ft/sec	Hammer
Initial						
38.75	35.25	45	6.8	30	12,589	Del Mag 30-23
Restrike						
38.75	35.25	45	6.8	29	12,589	

Anchor pile L was 38.75 ft E = 5127 ksi

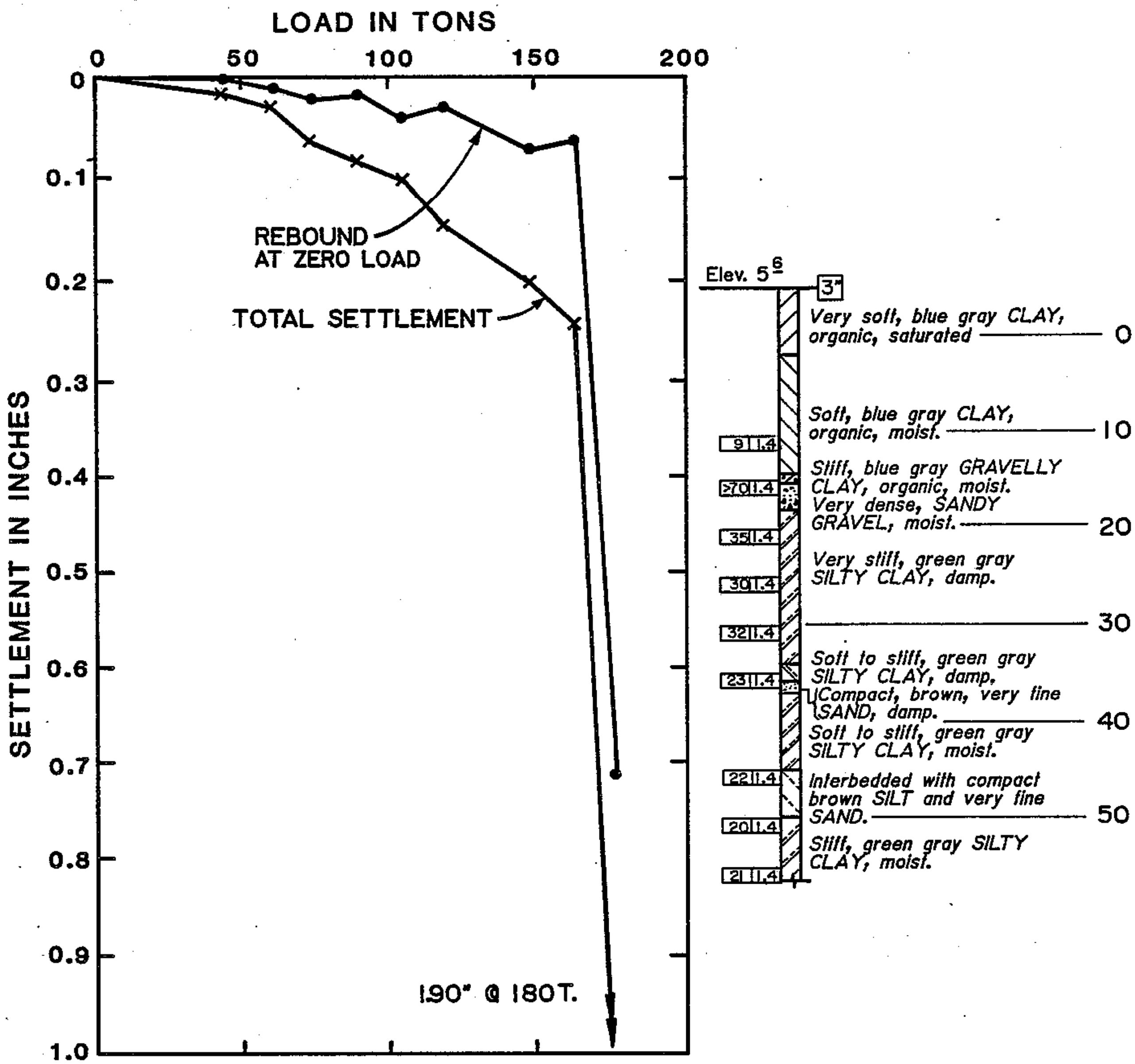
Initial Dynamic Test	75 Tons	$J_c = 0.28$
Restrike Dynamic Test	157 Tons (RMAX)	$J_c = 0.29$
Initial ENR 12 b/ft	41 Tons	
Restrike ENR 48 b/ft	128 Tons	
Static Load Test	165 Tons	Plunging failure
Anchor Pile Pullout	82 Tons	0.12" Uplift

Restrike Period - 18 hours

Static Load Period - 5 days after restrike

Penetration L - 37.25 ft

BEARING PILE TEST DATA
Bent 3, Bayview O.C., RICHMOND



STATIC LOAD TEST RESULTS AND LOG OF TEST BORING

Bent 3, Bayview O.C., RICHMOND

Restrike Bearing Pile

RTL	RMX	RSU	RMN	RAU
512	314	385	313	235
RSI	RS2	FMX	FT1	FT2
314	294	617	617	66
VMX	VT1	VT2	DMX	DT1
73	73	30	41	11
DT2	BPM	EMX	WUP	WDN
---	45	199	18	599
TMX	TMN	EA/c	Mc/L	2L/c
0	56	794	794	56
J_c	Δ	SFT	CTN	
29	0	99	-29	

DYNAMIC TEST RESULTS
Bent 3, Bayview O.C., RICHMOND

The following data for bearing pile are listed:

14 x 14 Prestressed Concrete Piles Specified Tip Elev. -70

L (ft)	L_g (ft)	BPM	Stroke (ft)	TR (%)	c ft/sec	Hammer
Initial						
67.0	64.0	45	6.8	35	12,219	Del Mag 30-23
Restrike						
67.0	64.0	42	7.8	21	12,219	

Anchor pile L was 77 ft

E = 4830 ksi

Initial Dynamic Test	136 Tons	J_c = 0.40
Restrike Dynamic Test	206 Tons (RMAX) 280 Tons (RSU)	J_c = 0.40
Initial ENR 16 b/ft	53 Tons	
Restrike ENR 70 b/6"	N/A	
Static Load Test	200 Tons	0.23" Settlement
Anchor Pile Pullout	100 Tons	0.40" Uplift

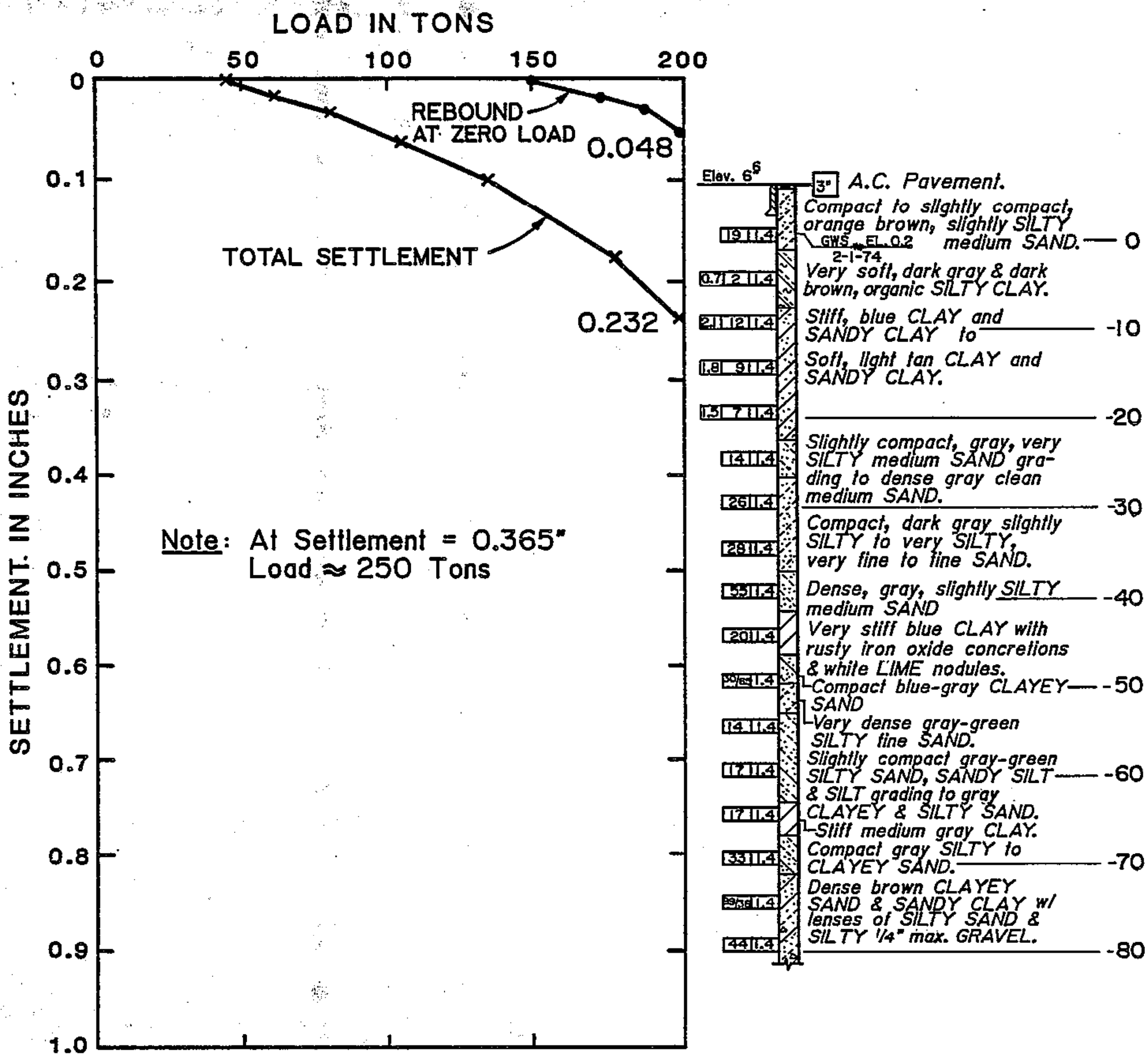
Restrike Period - 18 hours

Static Load Period - 8 days after restrike

Penetration L - 62 ft

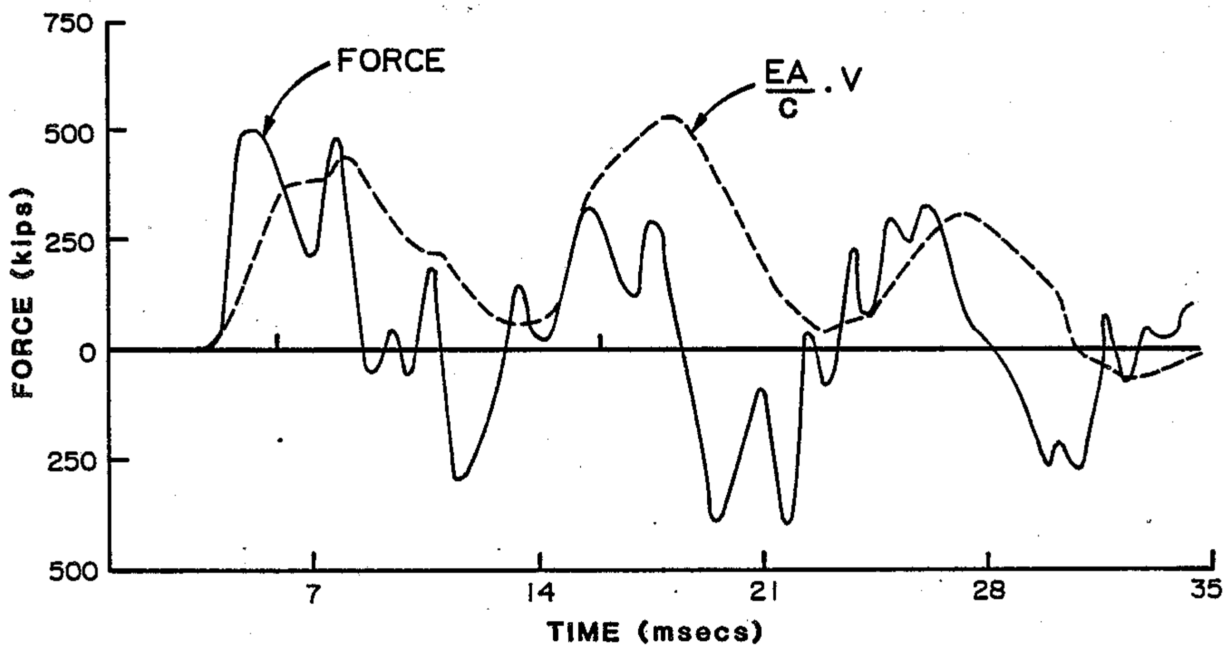
The first 40 ft wass predrilled with a 12" auger.

BEARING PILE TEST DATA
Bent 21, 380 Northbound Viaduct, SAN FRANCISCO AIRPORT

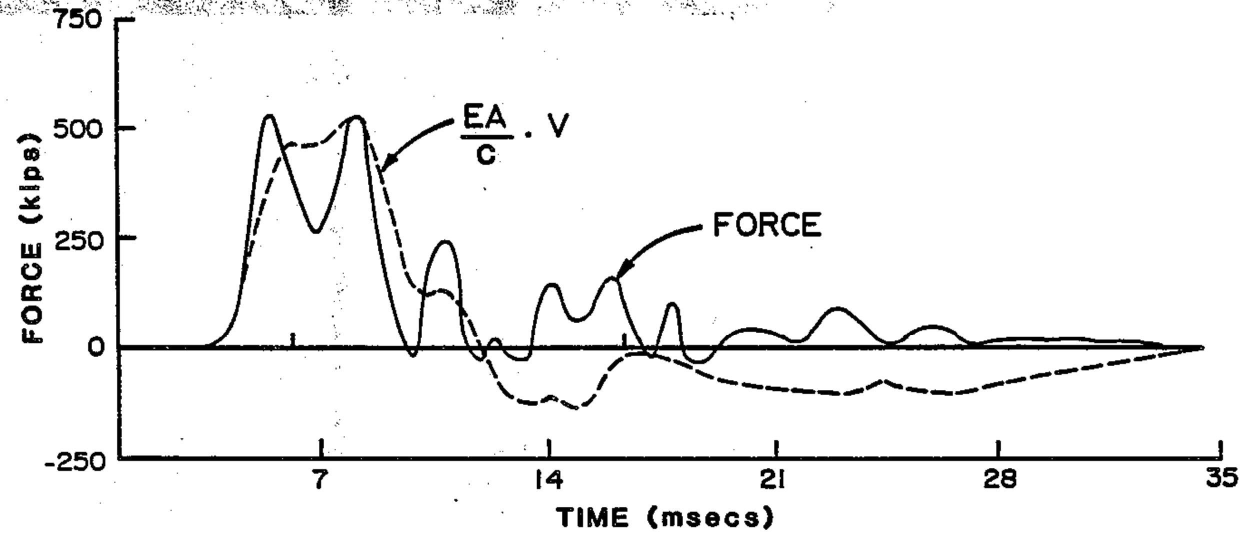


STATIC LOAD TEST RESULTS AND LOG OF TEST BORING

Bent 21, 380 Northbound Viaduct, SAN FRANCISCO AIRPORT



INITIAL WAVE TRACES
Bent 21, 380 Northbound Viaduct, SAN FRANCISCO AIRPORT



RESTRIKE WAVE TRACES
Bent 21, 380 Northbound Viaduct,
SAN FRANCISCO AIRPORT

The following data for bearing pile are listed:

14 x 14 Prestressed Concrete Pile Specified Tip Elev. -70

L (ft)	L_g (ft)	BPM	Stroke (ft)	TR (%)	c ft/sec	Hammer
Initial						
65.0	62.0	45	6.8	25	12,589	Del Mag 30-23
Restrike						
65.0	62.0	42	7.8	30	12,589	

Anchor pile L was 70 ft. E = 5127 ksi

Initial Dynamic Test	82 tons	J_c = 0.35
Restrike Dynamic Test	231 Tons (RMAX)	J_c = 0.35
Initial ENR 18 b/ft	59 Tons	
Restrike ENR 98 b/ft	234 Tons	
Static Load Test	220 Tons	0.39" Settlement
Anchor Pile Pullout	100 Tons	0.15" Uplift

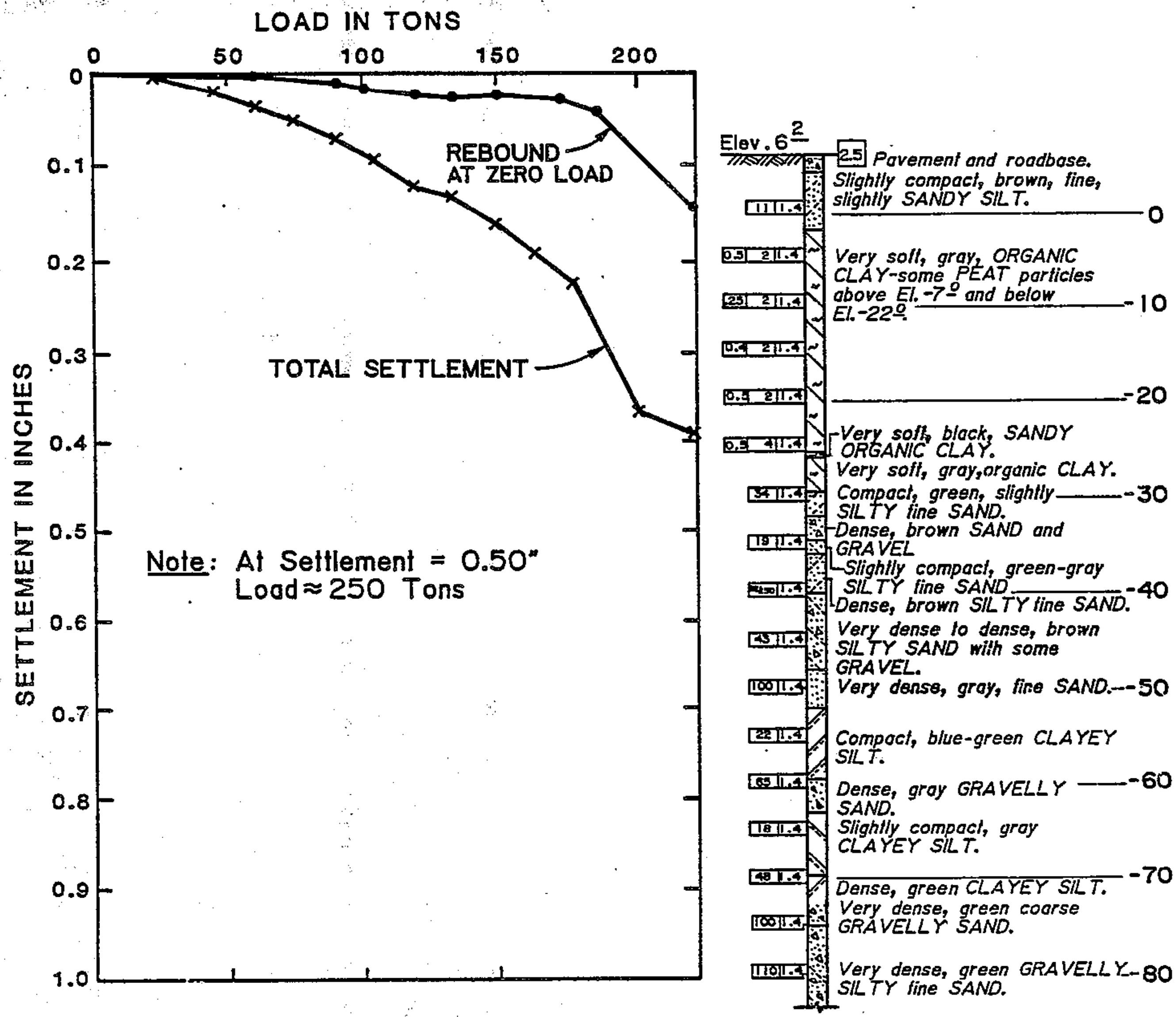
Restrike Period - 18 hours

Static Load Period - 13 days after restrike

Penetration L - 62 ft

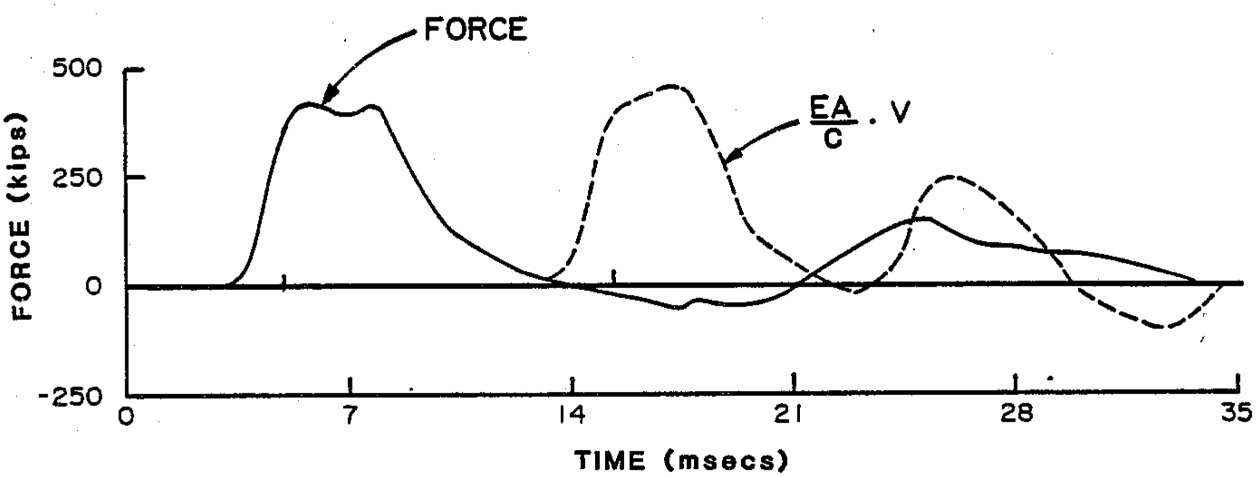
The first 40 ft was predrilled with a 12" auger

BEARING PILE TEST DATA
Bent 35, 380 Northbound Viaduct, SAN FRANCISCO AIRPORT

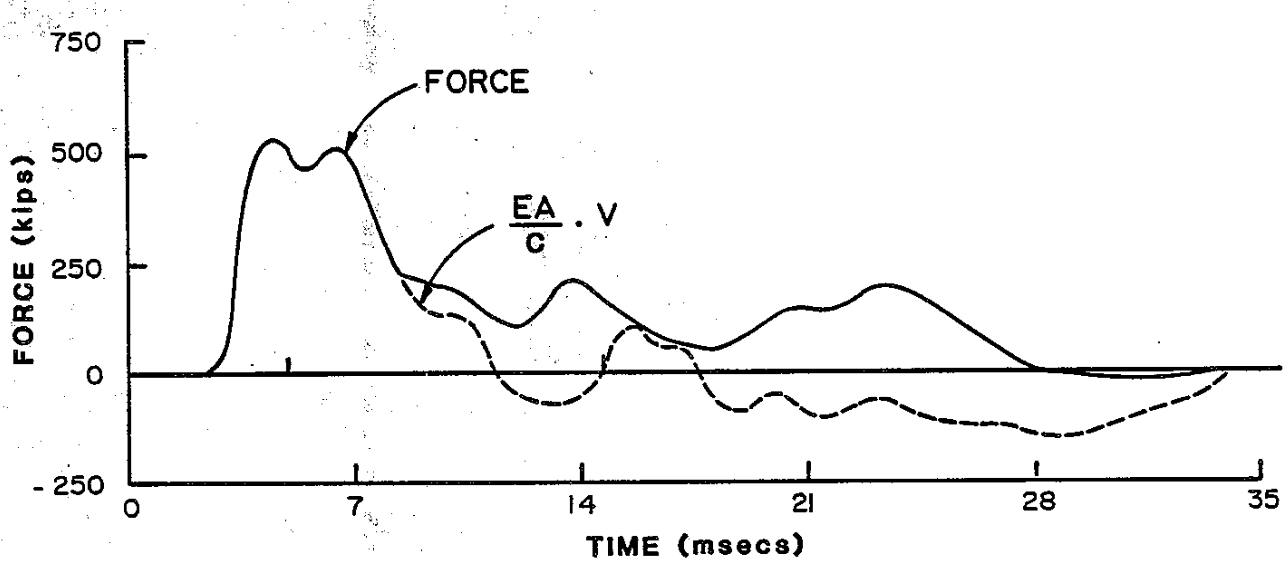


STATIC LOAD TEST RESULTS AND LOG OF TEST BORING

Bent 35, 380 Northbound Viaduct, SAN FRANCISCO AIRPORT



INITIAL WAVE TRACES
Bent 35, 380 Northbound Viaduct,
SAN FRANCISCO AIRPORT



RESTRIKE WAVE TRACES

Bent 35, 380 Northbound Viaduct, SAN FRANCISCO AIRPORT

The following data for bearing pile are listed:

15" Octagonal Prestressed Concrete Pile Specified Tip Elev. -48

L (ft)	L_g (ft)	BPM.	Stroke (ft)	TR (%)	c ft/sec	Hammer
Initial						
47.5	45.0	46	6.5	26	12,857	Del Mag 36-23
Restrike						
47.5	45.0	46	6.5	21	12,857	

Anchor pile L was 47.5 ft E = 5526 ksi

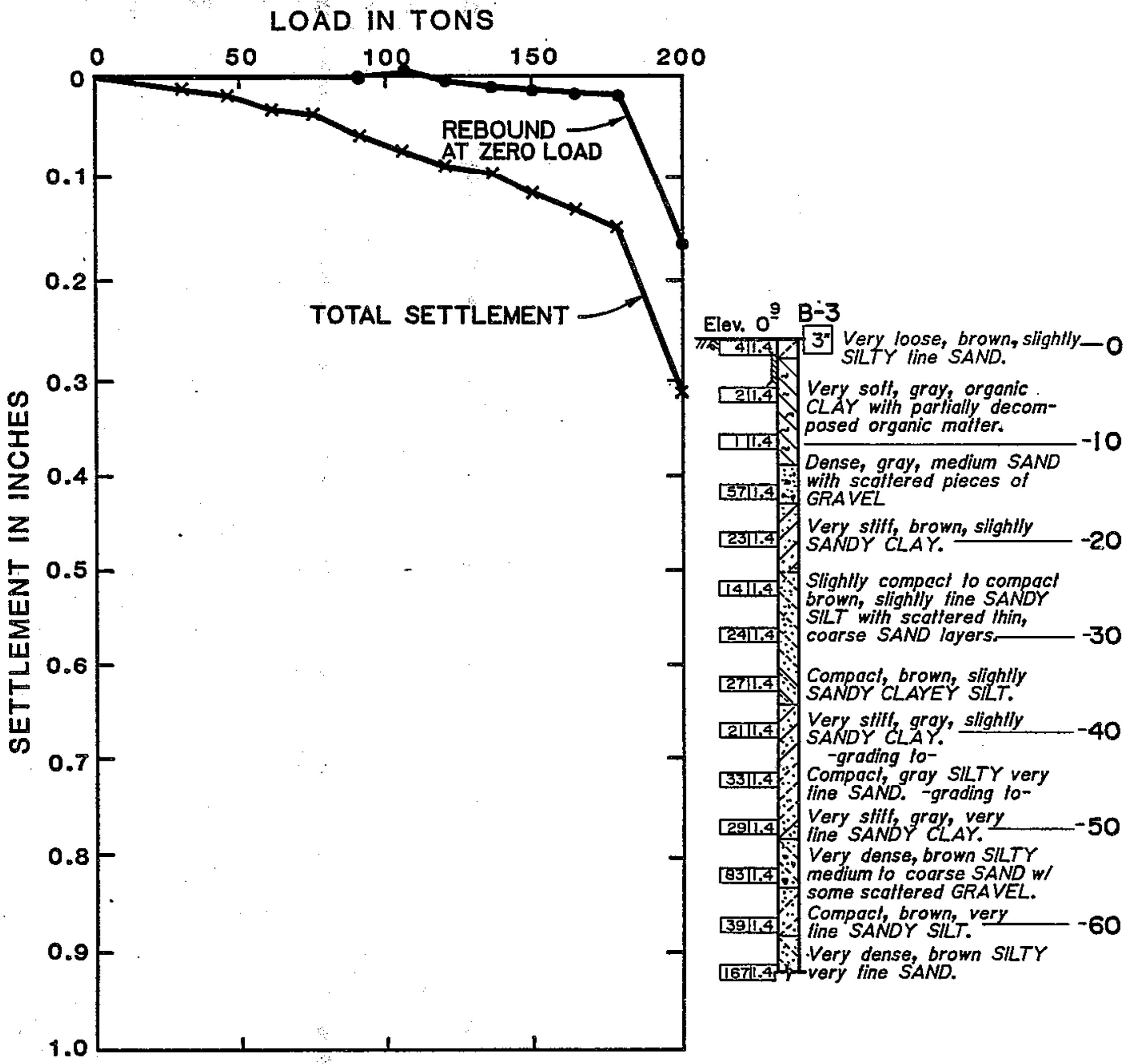
Initial Dynamic Test	108 Tons	J_c = 0.28
Restrike Dynamic Test	220 Tons (RMAX)	J_c = 0.28
Initial ENR 26 b/ft	92 Tons	
Restrike ENR 45 b/ft	143 Tons	
Static Load Test	200 Tons	0.306" Settlement
Anchor Pile Pullout	100 Tons	0.040" Uplift

Restrike Period - 18 hours

Static Load Period - 5 days after restrike

Penetration L - 44 ft

BEARING PILE TEST DATA
Bent 13, San Francisco Off-ramp,
SAN FRANCISCO AIRPORT



STATIC LOAD TEST RESULTS AND LOG OF TEST BORING

Bent 13, San Francisco On-Ramp SAN FRANCISCO AIRPORT

Restrike Bearing Pile

RTL	RMX	RSU	RMN	RAU
510	439	410	410	302
RSI	RS2	FMX	FT1	FT2
410	439	513	488	229
VMX	VT1	VT2	DMX	DT1
48	48	9	32	10
DT2	BPM	EMX	WUP	WDN
---	46	110	55	433
TMX	TMN	EA/c	Mc/L	2L/c
23	70	781	781	70
Jc	Δ	SFT	CTN	
28	0	283	0	

DYNAMIC TEST RESULTS
Bent 13, San Francisco Off-Ramp, SAN FRANCISCO AIRPORT

The following data for bearing pile are listed:

15 inch octagonal prestressed concrete pile Specified Tip Elev. -52

L (ft)	L_g (ft)	BPM	Stroke (ft)	TR (%)	c ft/sec	Hammer
Initial						
51.7	49.2	40	8.7	19	12,300	Del Mag 36-23
Restrike						
51.7	49.2	43	7.5	26	12,300	

Anchor pile L was 47.6 ft E = 4894 ksi

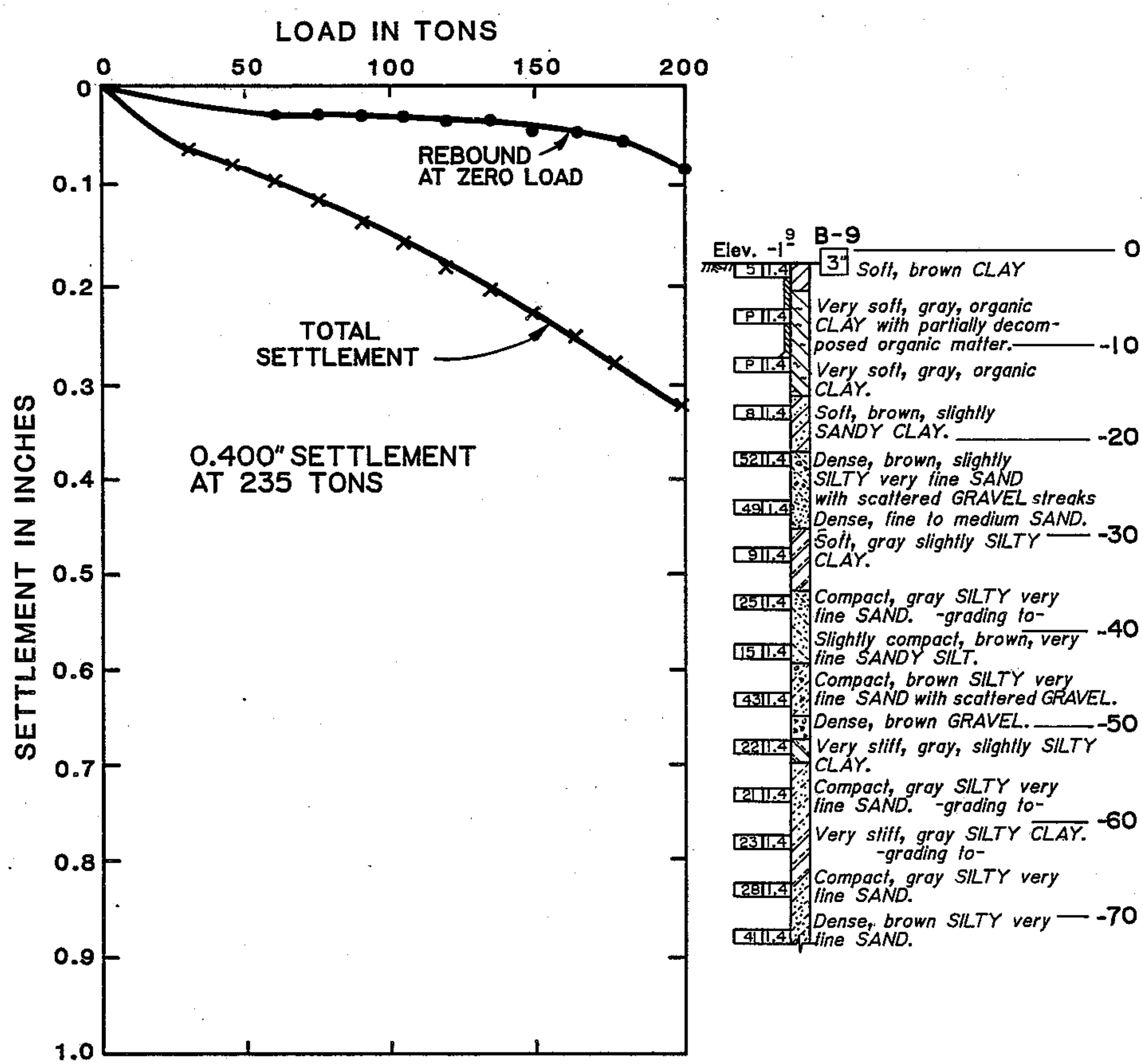
Initial Dynamic Test	150 Tons	J_c = 0.28
Restrike Dynamic Test	234 Tons (RMAX)	J_c = 0.28
Initial ENR 10 b/ft	53 Tons	
Restrike ENR 41 b/ft	152 Tons	
Static Load Test	235 Tons	0.40 Settlement
Anchor Pile Pullout	100 Tons	0.18 Uplift

Restrike Period - 18 hours

Static Load Period - 1 day after restrike

Penetration L - 48.7 ft

BEARING PILE TEST DATA
Bent 12, San Francisco On-Ramp, SAN FRANCISCO AIRPORT



STATIC LOAD TEST RESULTS AND LOG OF TEST BORING

Bent 12, San Francisco Off-Ramp SAN FRANCISCO AIRPORT

Restrike Bearing Pile

RTL	RMX	RSU	RMN	RAU
467	468	332	332	321
RSI	RS2	FMX	FT1	FT2
332	---	494	490	119
VMX	VT1	VT2	DMX	DT1
64	64	21	54	13
DT2	BPM	EMX	WUP	WDN
---	43	153	6	485
TMX	TMN	EA/c	Mc/L	2L/c
34	80	749	749	80
Jc	Δ	SFT	CTN	
27	0	-377	0	

DYNAMIC TEST RESULTS
Bent 12, San Francisco On-Ramp, SAN FRANCISCO

The following data for bearing pile are listed:

15" Octagonal Prestressed Concrete Pile   Specified Tip Elev. -47

L (ft)	L_g (ft)	BPM	Stroke (ft)	TR (%)	c ft/sec	Hammer
Initial						
48.17	45.67	56	4.3	N/A	11,710	FEC 3000
Restrike						
48.17	45.67	42	5.5	28	11,710	

Anchor pile L was 52 ft   E = 4436 ksi

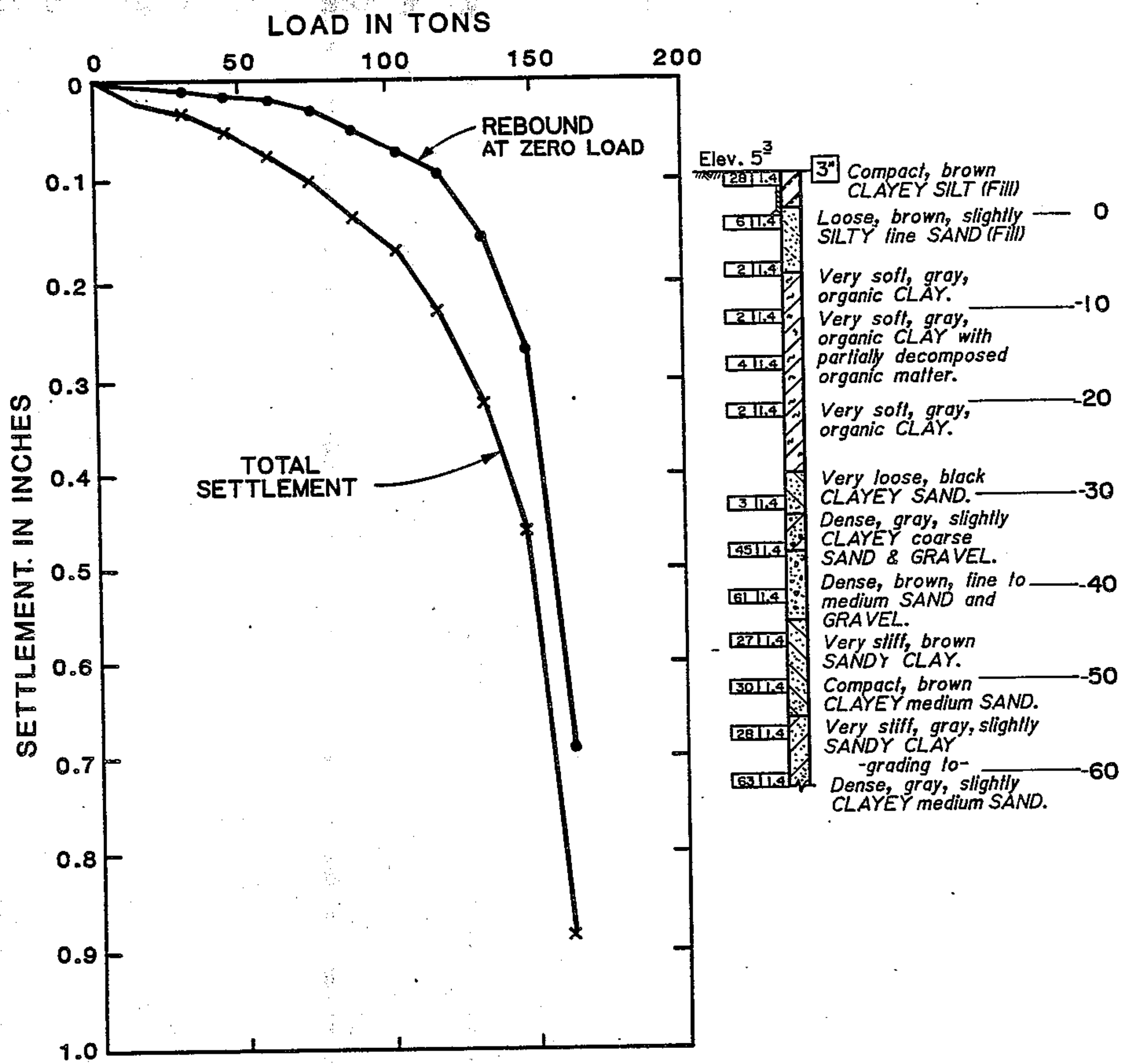
Initial Dynamic Test		72 Tons	$J_c = 0.30$
Restrike Dynamic Test		136 Tons (RMAX)	$J_c = 0.28$
Initial ENR	25 b/ft	49 Tons	
Restrike ENR	42 b/ft	93 Tons	
Static Load Test		150 Tons	Plunging failure
Anchor Pile Pullout		75 Tons	0.20" Uplift

Restrike Period - 18 hours

Static Load Period - 6 days after restrike

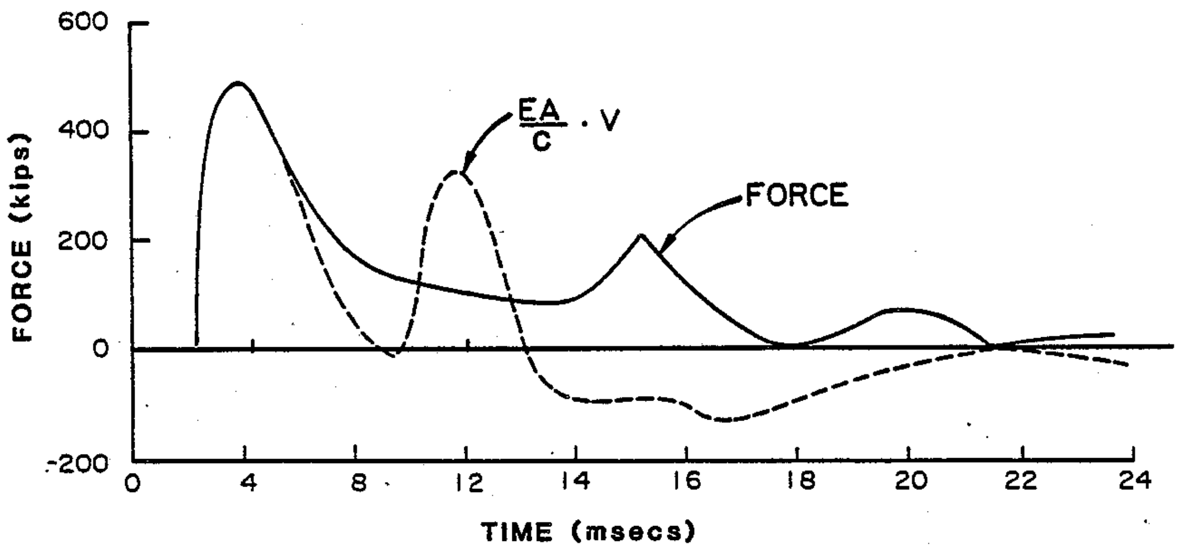
Penetration L - 45 ft

BEARING PILE TEST DATA
Bent 15, San Francisco Off-Ramp
SAN FRANCISCO AIRPORT



STATIC LOAD TEST RESULTS AND LOG OF TEST BORING

Bent 15, San Francisco Off-ramp, SAN FRANCISCO AIRPORT



RESTRIKE WAVE TRACES

Bent 15, San Francisco Off-Ramp
SAN FRANCISCO AIRPORT

The following data for bearing pile are listed:

10HP57 Steel Pile Specified Tip Elev. -13

L (ft)	L_g (ft)	BPM	Stroke (ft)	TR (%)	c ft/sec	Hammer
Initial						
60	58	50	5.5	50	16,800	MKT 70B
Restrike						
None						

Anchor pile L was 60 ft E = 29,826 ksi

Initial Dynamic Test 164 Tons (RMAX) J_c = 0.10

Restrike Dynamic Test

Initial ENR 20 b/ft 55 Tons

Restrike ENR N/A

Static Load Test 140 Tons 0.10" Settlement

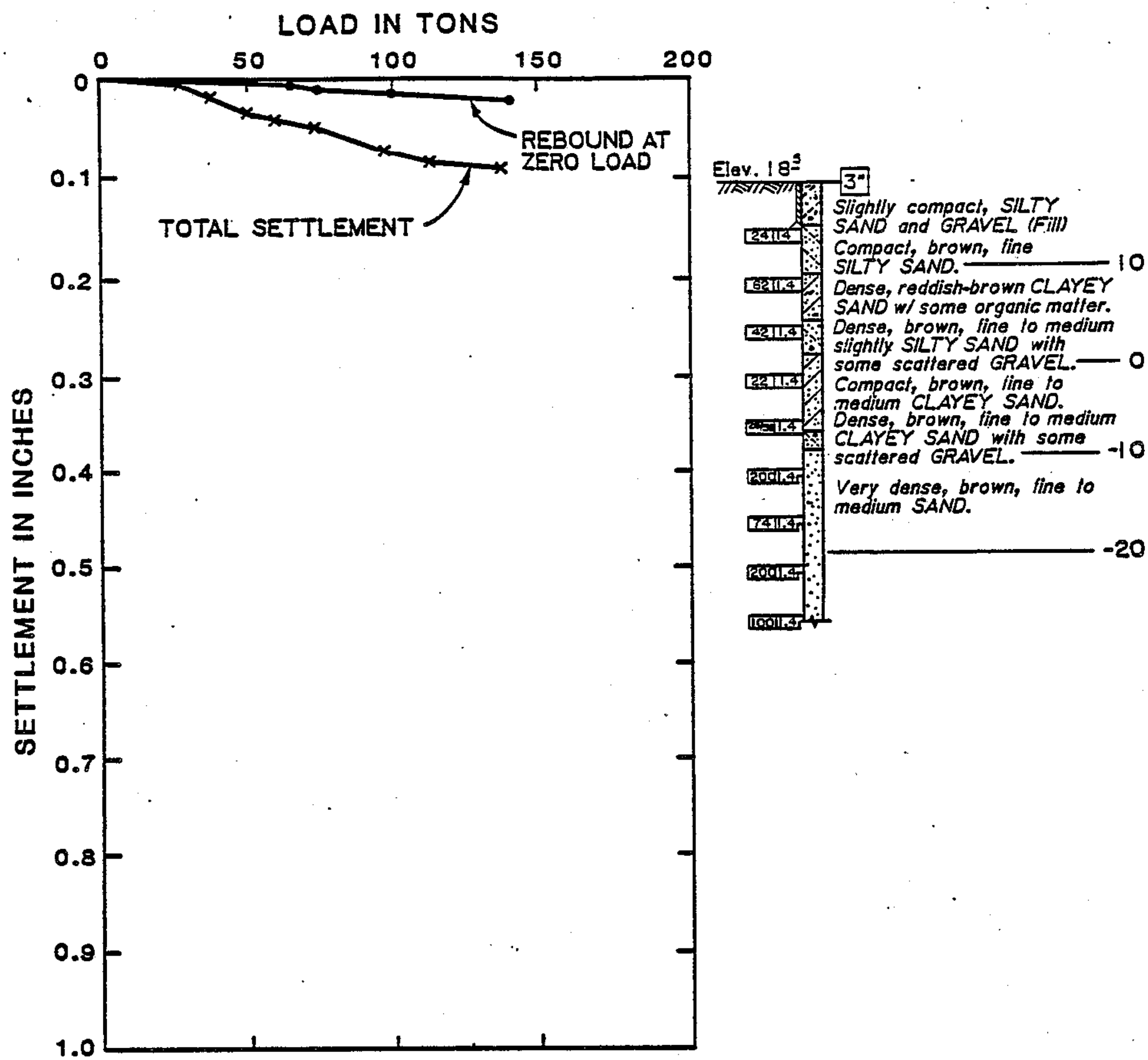
Anchor Pile Pullout 70 Tons 0.07" Uplift

Restrike Period - None

Static Load Period - 7 days after initial drive

Penetration L - 30 ft

BEARING PILE TEST DATA
Bent 8, Adeline St. Viaduct, OAKLAND



STATIC LOAD TEST RESULTS & LOG OF TEST BORING

Bent 8, Adeline St. Viaduct, OAKLAND

The following data for bearing pile are listed:

10HP57 Steel Pile Specified Tip Elev. -15

L (ft)	L_g (ft)	BPM	Stroke (ft)	TR (%)	c ft/sec	Hammer
Initial						
60	58	51	5.3	37	16,800	MKT 70B
Restrike						
None						

Anchor pile L was 60 ft E = 29,826 ksi

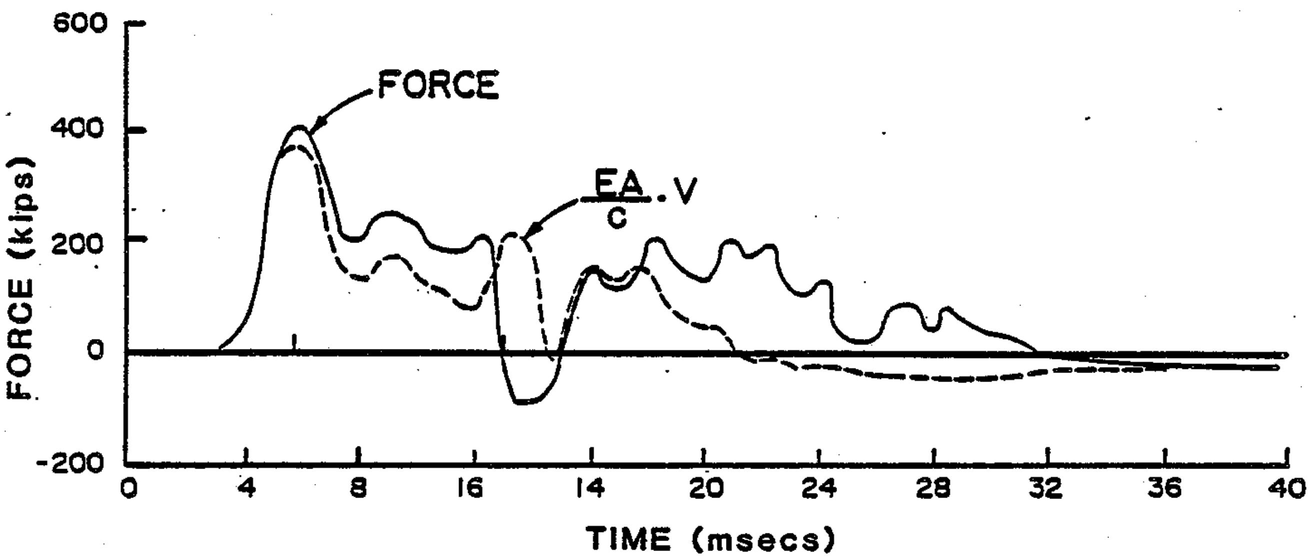
Initial Dynamic Test	124 Tons (RMAX) $J_c = 0.10$	
Restrike Dynamic Test		
Initial ENR 17b/ft	46 Tons	
Restrike ENR		
Static Load Test	140 Tons	0.10" Settlement
Anchor Pile Pullout	70 Tons	0.04" Uplift

Restrike Period - None

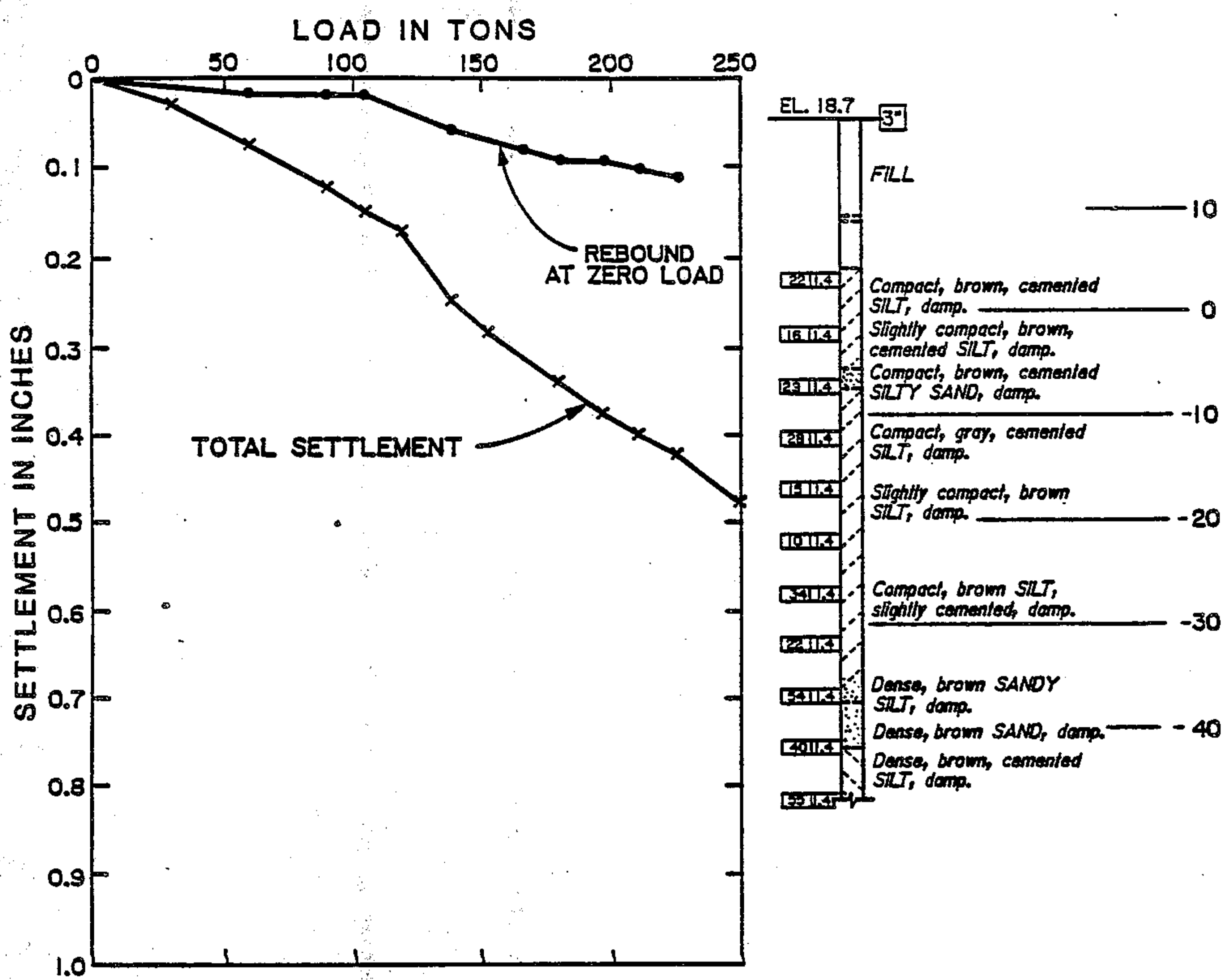
Static Load Period - 13 days after initial drive

Penetration L - 27.6 ft

BEARING PILE TEST DATA
Bent 5, Madison St. U.C., OAKLAND



INITIAL WAVE TRACES
Bent 5, Madison St. U.C., OAKLAND



STATIC LOAD TEST RESULTS AND LOG OF TEST BORING

Bent 29L, Crosstown Viaduct, STOCKTON

The following data for bearing pile are listed:

18 x 18 Prestressed Concrete Pile Specified Tip Elev. -75

L (ft)	L_g (ft)	BPM	Stroke (ft)	TR (%)	c ft/sec	Hammer
Initial						
85	82	40	8.7	29	12,193	Del Mag 36-23
Restrike						
85	82	41	8.3	28	12,193	

Anchor pile L was 87 ft E = 4809 ksi

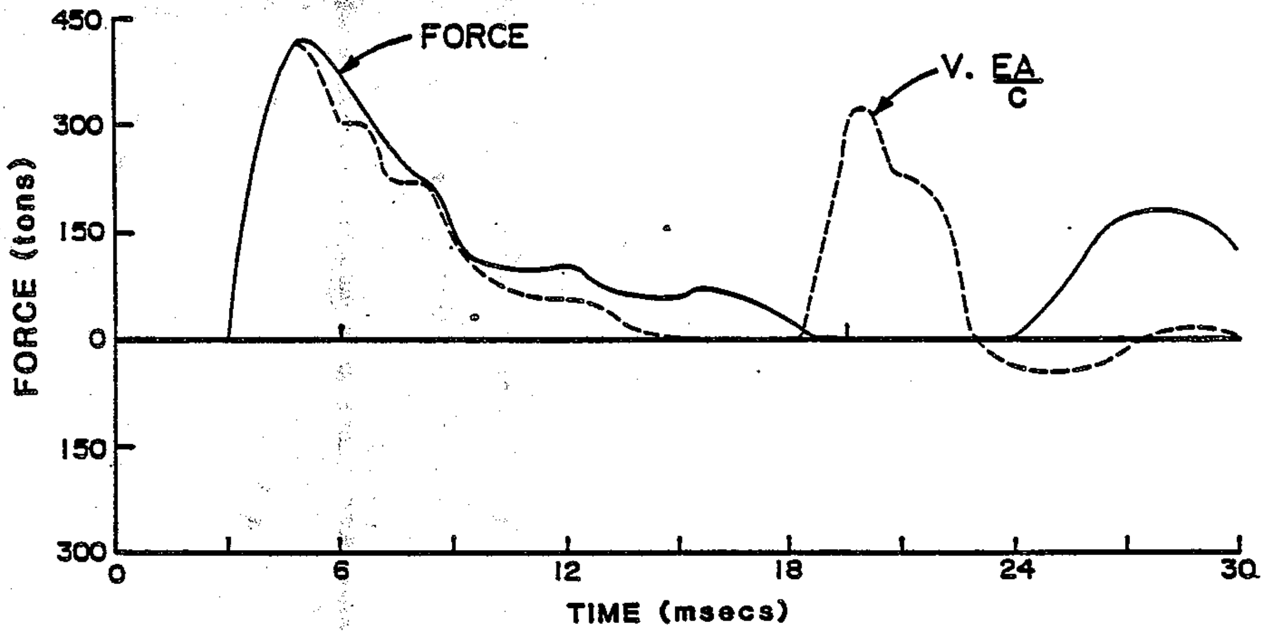
Initial Dynamic Test	185 Tons (RMAX)	$J_c = 0.45$
Restrike Dynamic Test	309 Tons (RMAX) 274 (RMN) 425 (RSU)	$J_c = 0.45$
Initial ENR 32 b/ft	144 Tons	
Restrike ENR 25 b/1 3/4"	N/A	
Static Load Test	250 Tons	0.40" Settlement
Anchor Pile Pullout	125 Tons	0.31" Uplift

Restrike Period - 3 days

Static Load Period - 12 days after restrike

Penetration L - 82 ft

BEARING PILE TEST DATA
Abutment 1, Sweetwater Channel Bridge, SAN DIEGO



INITIAL WAVE TRACES
Abutment 1, Sweetwater Channel Bridge,
SAN DIEGO

The following data for bearing pile are listed:

12 x 12 Prestressed Concrete Pile Specified Tip Elev. -15

L (ft)	L_g (ft)	BPM	Stroke (ft)	TR (%)	c ft/sec	Hammer
Initial						
32.25	29.75	43	7.5	27	12,396	Del Mag 30-23
Restrike						
32.25	29.75	41	8.3	19	12,396	
Anchor pile L was 37.25 ft					E =4971 ksi	

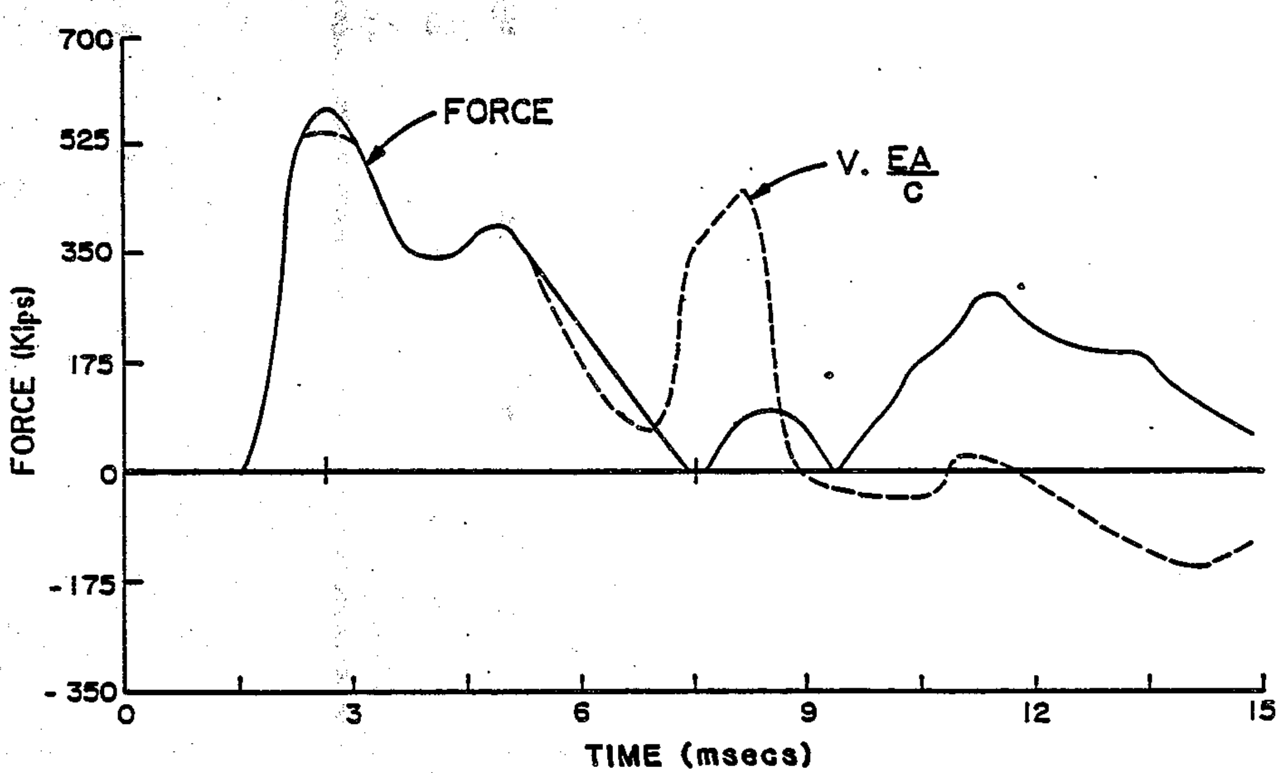
Initial Dynamic Test	175 Tons	$J_c = 0.25$
Restrike Dynamic Test	234 Tons (RMAX)	$J_c = 0.25$
Initial ENR 40 b/ft	124 Tons	
Restrike ENR (72 b/ft)	N/A	
Static Load Test	240 Tons	0.36" Settlement
Anchor Pile Pullout	120 Tons	0.53" Uplift

Restrike Period - 18 hours

Static Load Period - 3 days after restrike

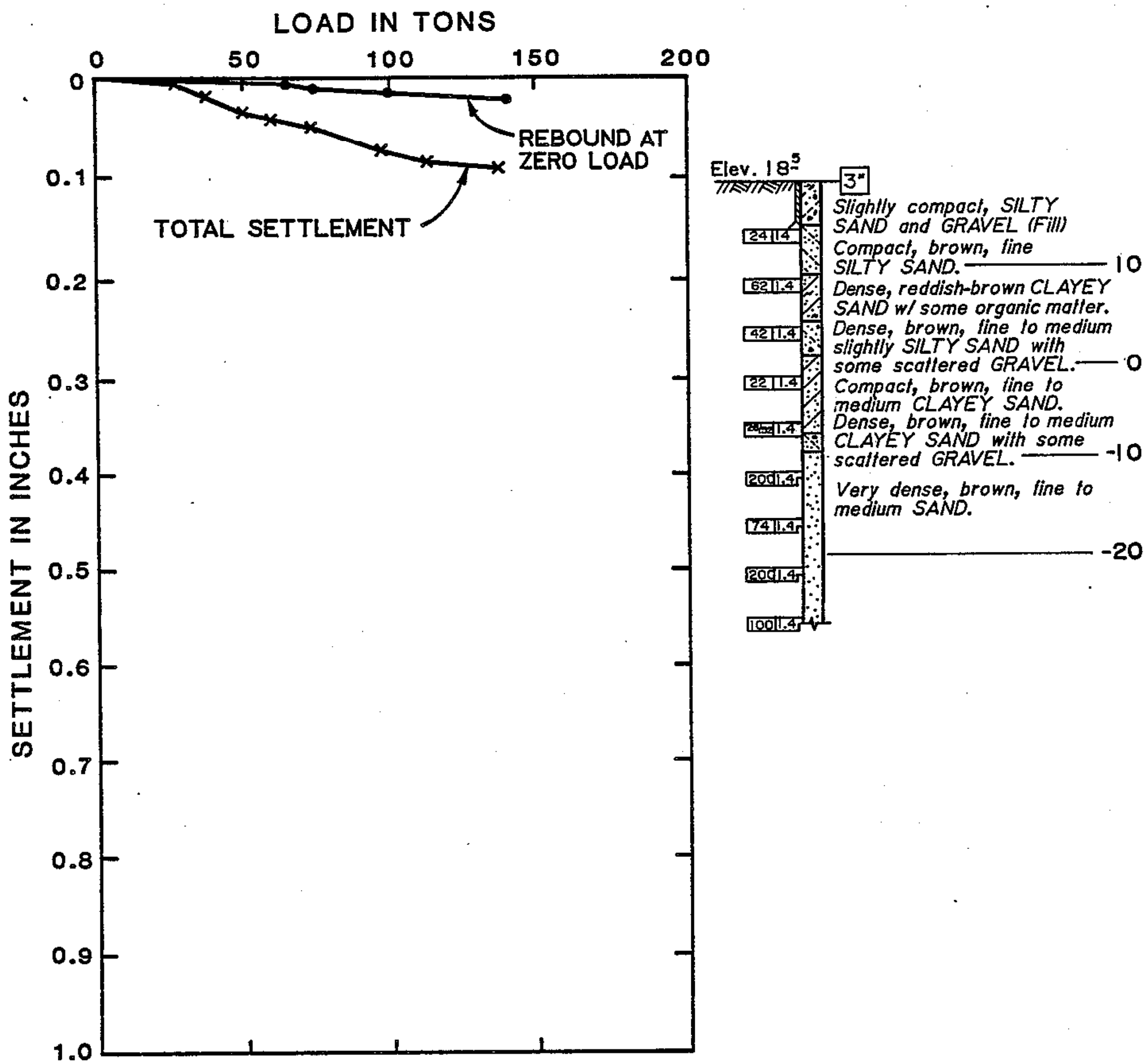
Penetration L - 26.75 ft

BEARING PILE TEST DATA
Bent 4, Sacramento Light Rail Transit, SACRAMENTO



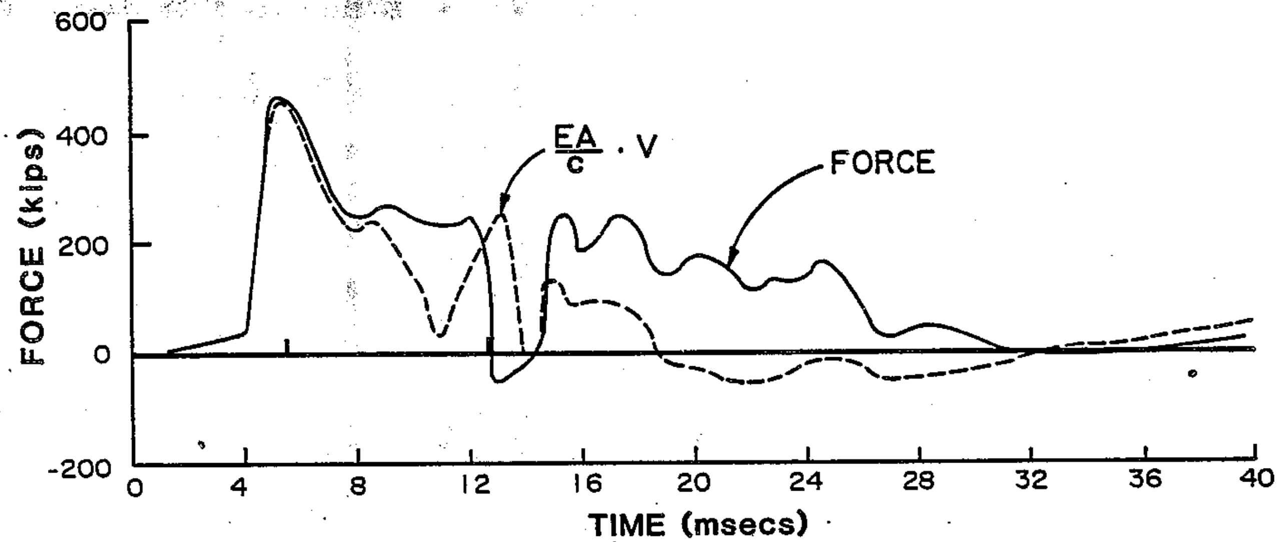
INITIAL WAVE TRACES

Bent 4, Sacramento Light Rail Transit, SACRAMENTO



STATIC LOAD TEST RESULTS & LOG OF TEST BORING

Bent 8, Adeline St. Viaduct, OAKLAND



INITIAL WAVE TRACES
Bent 8, Adeline St. Viaduct, OAKLAND

The following data for bearing pile are listed:

10HP57 Steel Pile Specified Tip Elev. -15

L (ft)	L_g (ft)	BPM	Stroke (ft)	TR (%)	c ft/sec	Hammer
Initial						
60	58	51	5.3	37	16,800	MKT 70B
Restrike						
None						

Anchor pile L was 60 ft E = 29,826 ksi

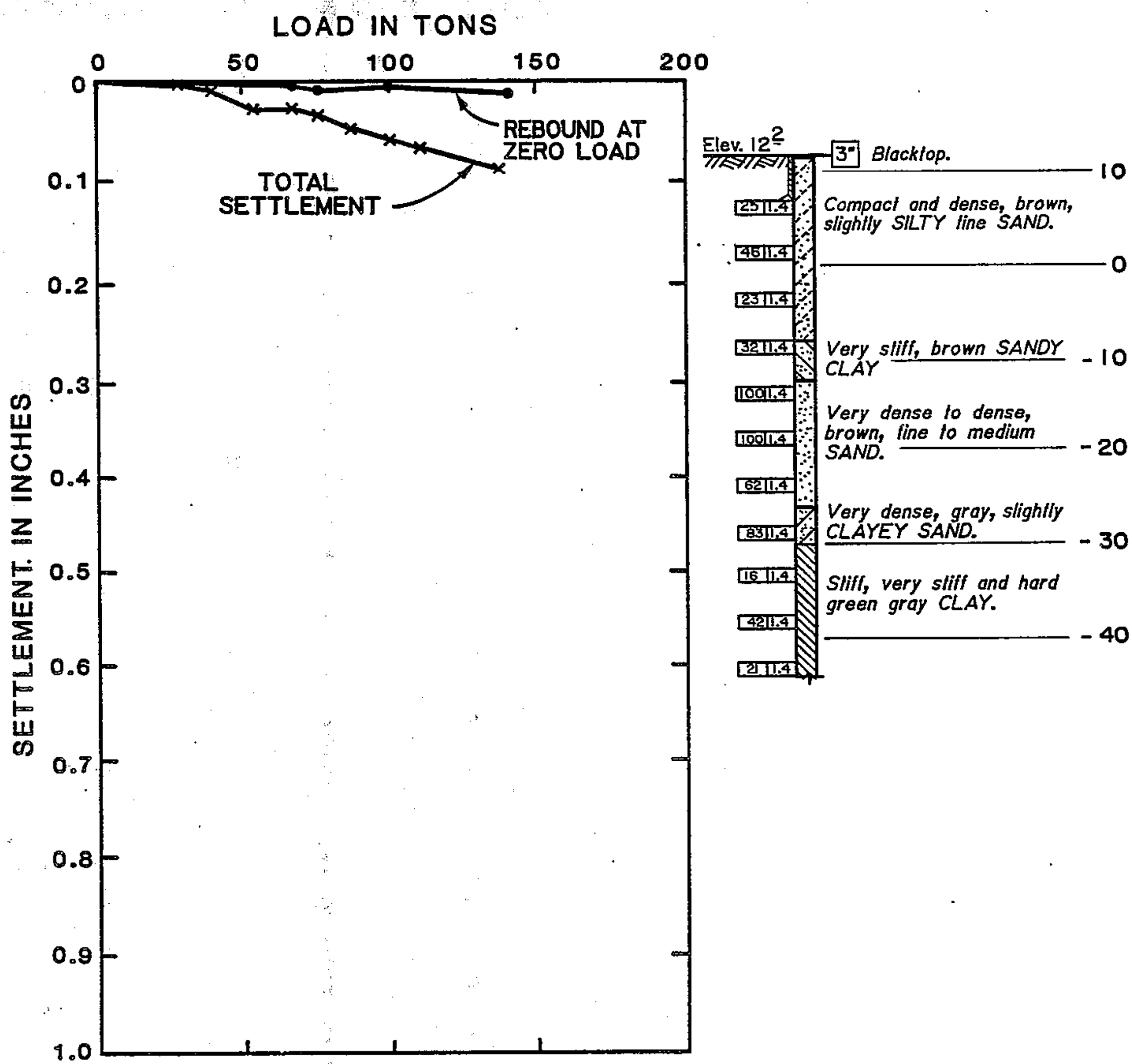
Initial Dynamic Test	124 Tons (RMAX) $J_c = 0.10$	
Restrike Dynamic Test		
Initial ENR 17b/ft	46 Tons	
Restrike ENR		
Static Load Test	140 Tons	0.10" Settlement
Anchor Pile Pullout	70 Tons	0.04" Uplift

Restrike Period - None

Static Load Period - 13 days after initial drive

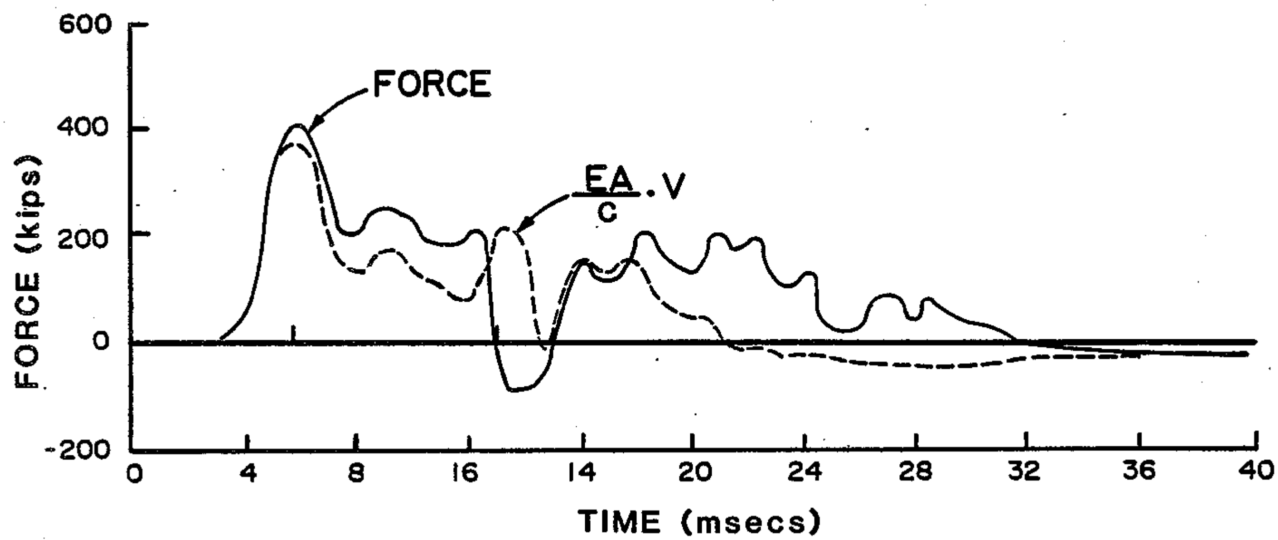
Penetration L - 27.6 ft

BEARING PILE TEST DATA
Bent 5, Madison St. U.C., OAKLAND



STATIC LOAD TEST RESULTS AND LOG OF TEST BORING

Bent 5, Madison St. U.C., OAKLAND



INITIAL WAVE TRACES
Bent 5, Madison St. U.C., OAKLAND

The following data for bearing pile are listed:

12" x 12" Prestressed Concrete Pile   Specified Tip Elev. -40

L (ft)	L_g (ft)	BPM	Stroke (ft)	TR (%)	c ft/sec	Hammer
Initial						
		N/A	N/A	N/A		
Restrike						
59.25	56.3	44	7.1	30	12,500	Del Mag 36-23

Anchor pile L was 56.3 ft   E = 5055 ksi

Initial Dynamic Test	N/A	
Restrike Dynamic Test	250 Tons (RMAX) J_c = 0.45	
Initial ENR 8 b/ft	N/A	
Restrike ENR 30 b/ft	112 Tons	
Static Load Test	250 Tons	0.47" Settlement
Anchor Pile Pullout	125 Tons	0.21" Uplift

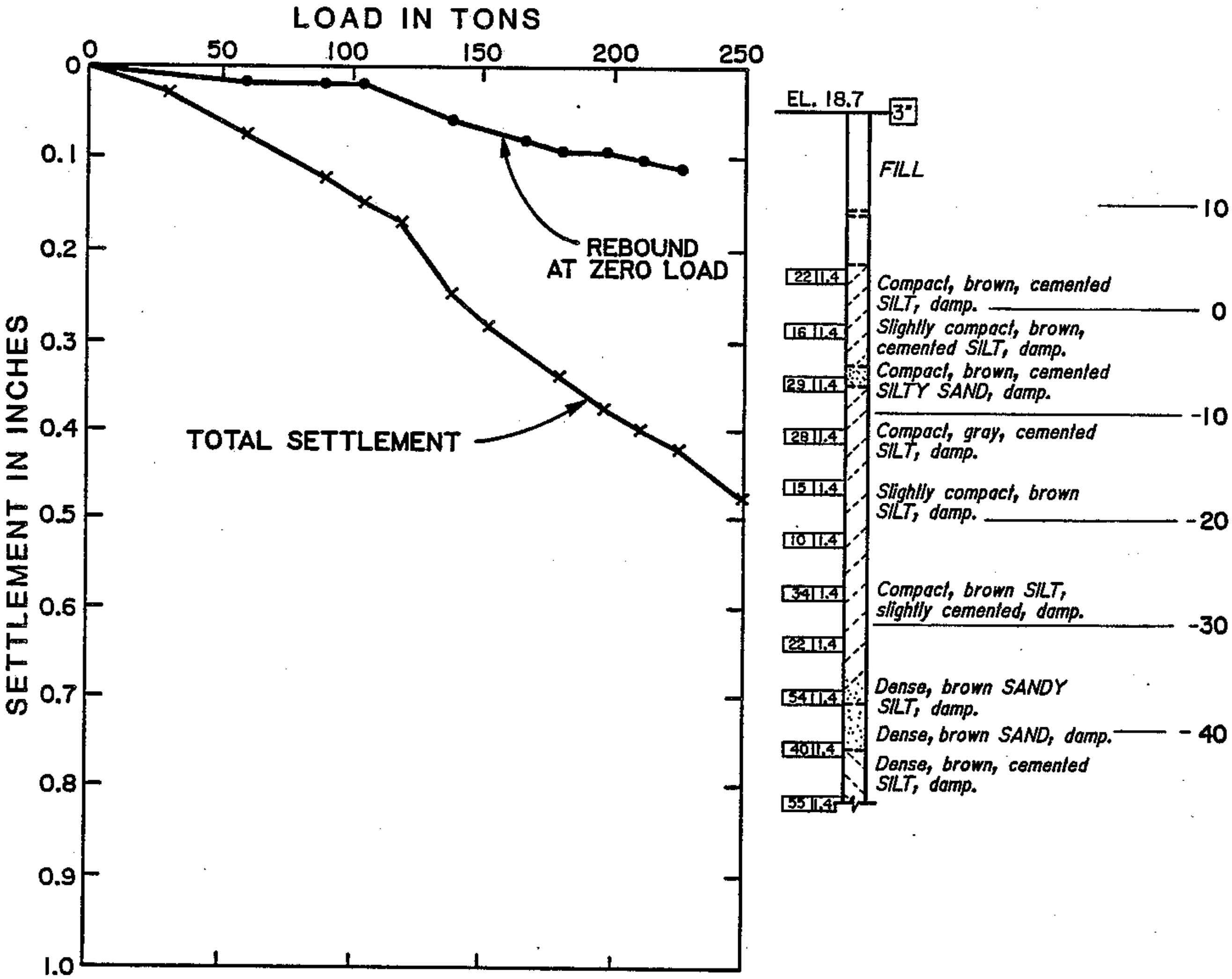
Restrike Period - 18 hours

Static Load Period - 20 days after restrike

Penetration L - 56.3 ft

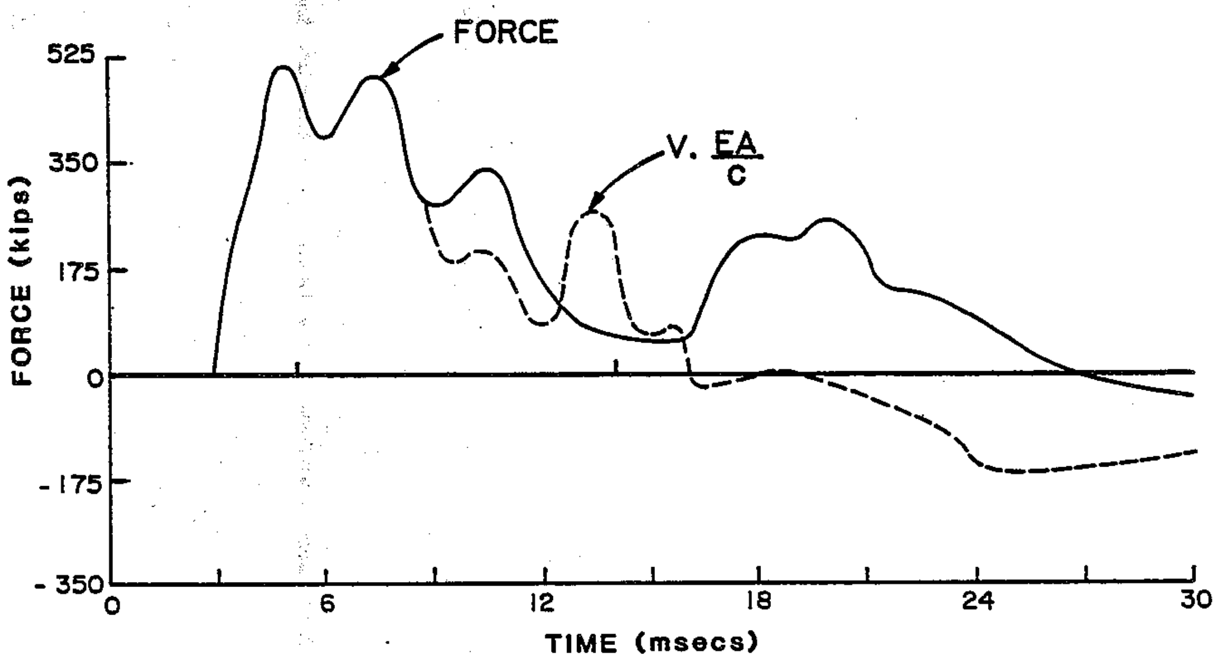
The first 38 ft was predrilled with a 12" auger

BEARING PILE TEST DATA
Bent 29L, Crosstown Viaduct, STOCKTON



STATIC LOAD TEST RESULTS AND LOG OF TEST BORING

Bent 29L, Crosstown Viaduct, STOCKTON



RESTRIKE WAVE TRACES
Bent 29L, Crosstown Viaduct, STOCKTON

The following data for bearing pile are listed:

18 x 18 Prestressed Concrete Pile   Specified Tip Elev. -75

L (ft)	L_g (ft)	BPM	Stroke (ft)	TR (%)	c ft/sec	Hammer
Initial						
85	82	40	8.7	29	12,193	Del Mag 36-23
Restrike						
85	82	41	8.3	28	12,193	

Anchor pile L was 87 ft   E = 4809 ksi

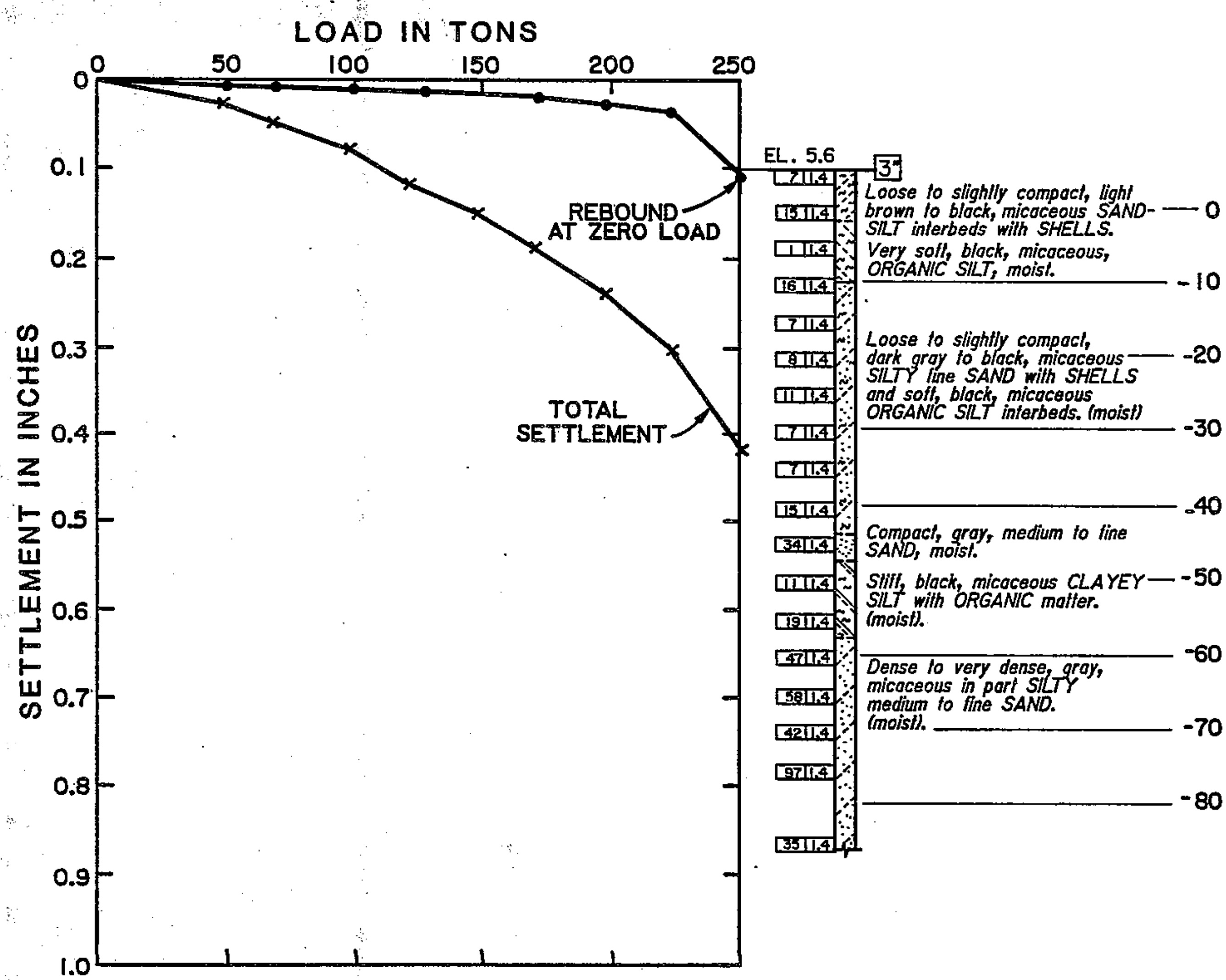
Initial Dynamic Test	185 Tons (RMAX)	J_c = 0.45
Restrike Dynamic Test	309 Tons (RMAX) 425 (RSU)	274 (RMN) J_c = 0.45
Initial ENR 32 b/ft	144 Tons	
Restrike ENR 25 b/1 3/4"	N/A	
Static Load Test	250 Tons	0.40" Settlement
Anchor Pile Pullout	125 Tons	0.31" Uplift

Restrike Period - 3 days

Static Load Period - 12 days after restrike

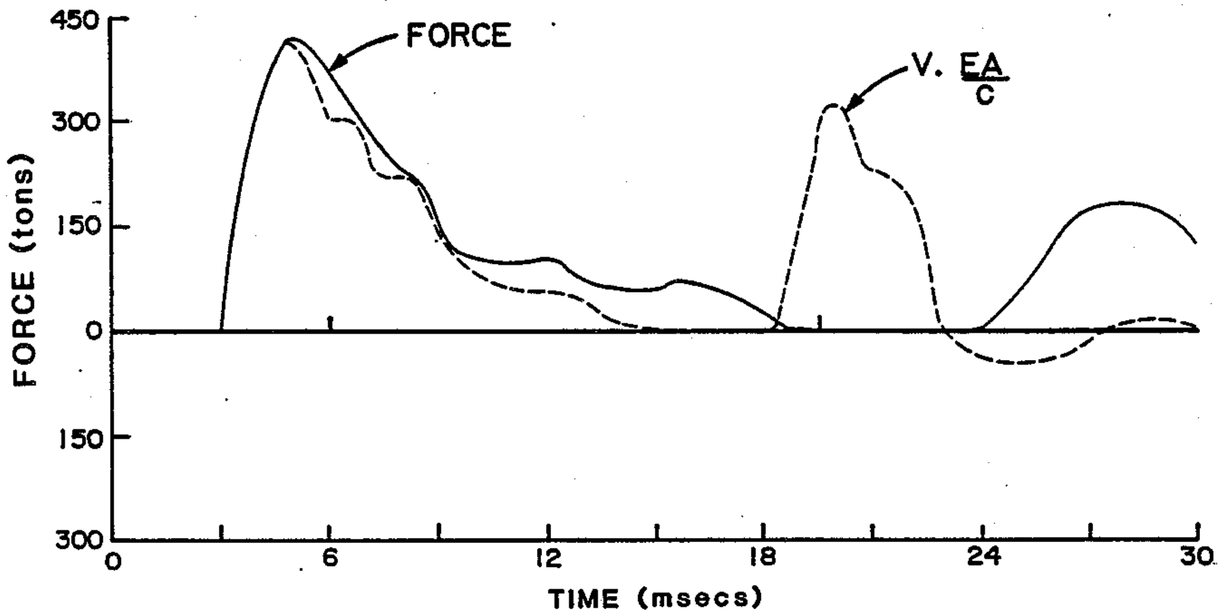
Penetration L - 82 ft

BEARING PILE TEST DATA
Abutment 1, Sweetwater Channel Bridge, SAN DIEGO



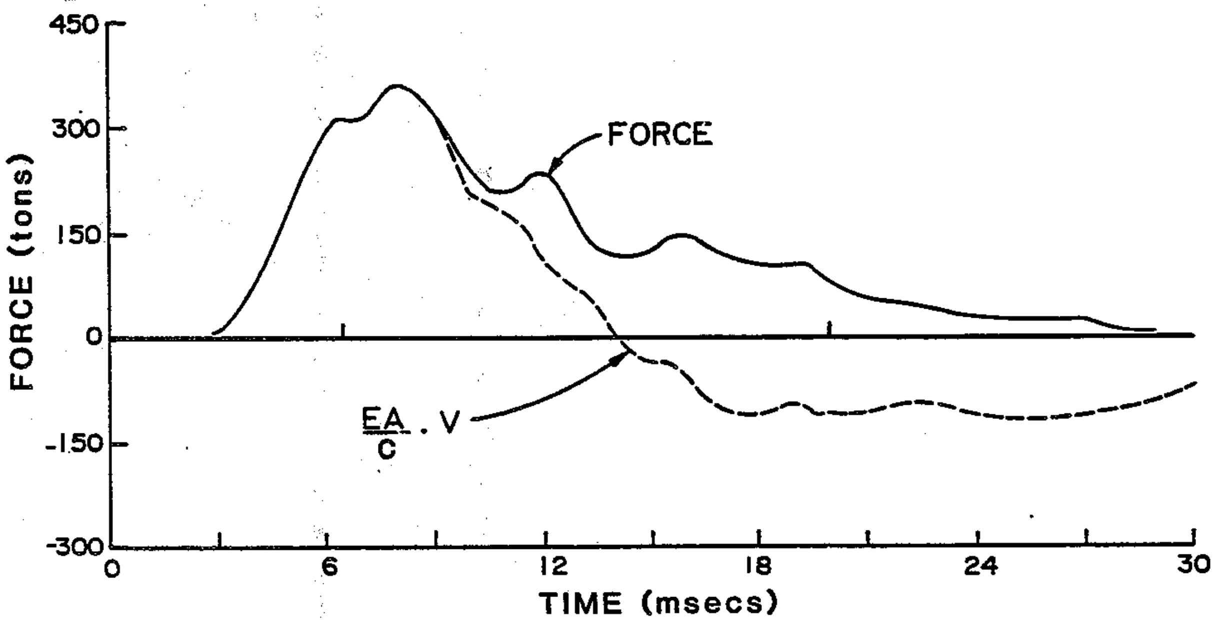
STATIC LOAD TEST RESULTS AND LOG OF TEST BORING

Abutment 1, Sweetwater Channel Bridge, SAN DIEGO



INITIAL WAVE TRACES

Abutment 1, Sweetwater Channel Bridge, SAN DIEGO



RESTRIKE WAVE TRACES

Abutment 1, Sweetwater Channel Bridge,
SAN DIEGO

The following data for bearing pile are listed:

12 x 12 Prestressed Concrete Pile Specified Tip Elev. -15

L (ft)	L_g (ft)	BPM	Stroke (ft)	TR (%)	c ft/sec	Hammer
Initial						
32.25	29.75	43	7.5	27	12,396	Del Mag 30-23
Restrike						
32.25	29.75	41	8.3	19	12,396	

Anchor pile L was 37.25 ft E =4971 ksi

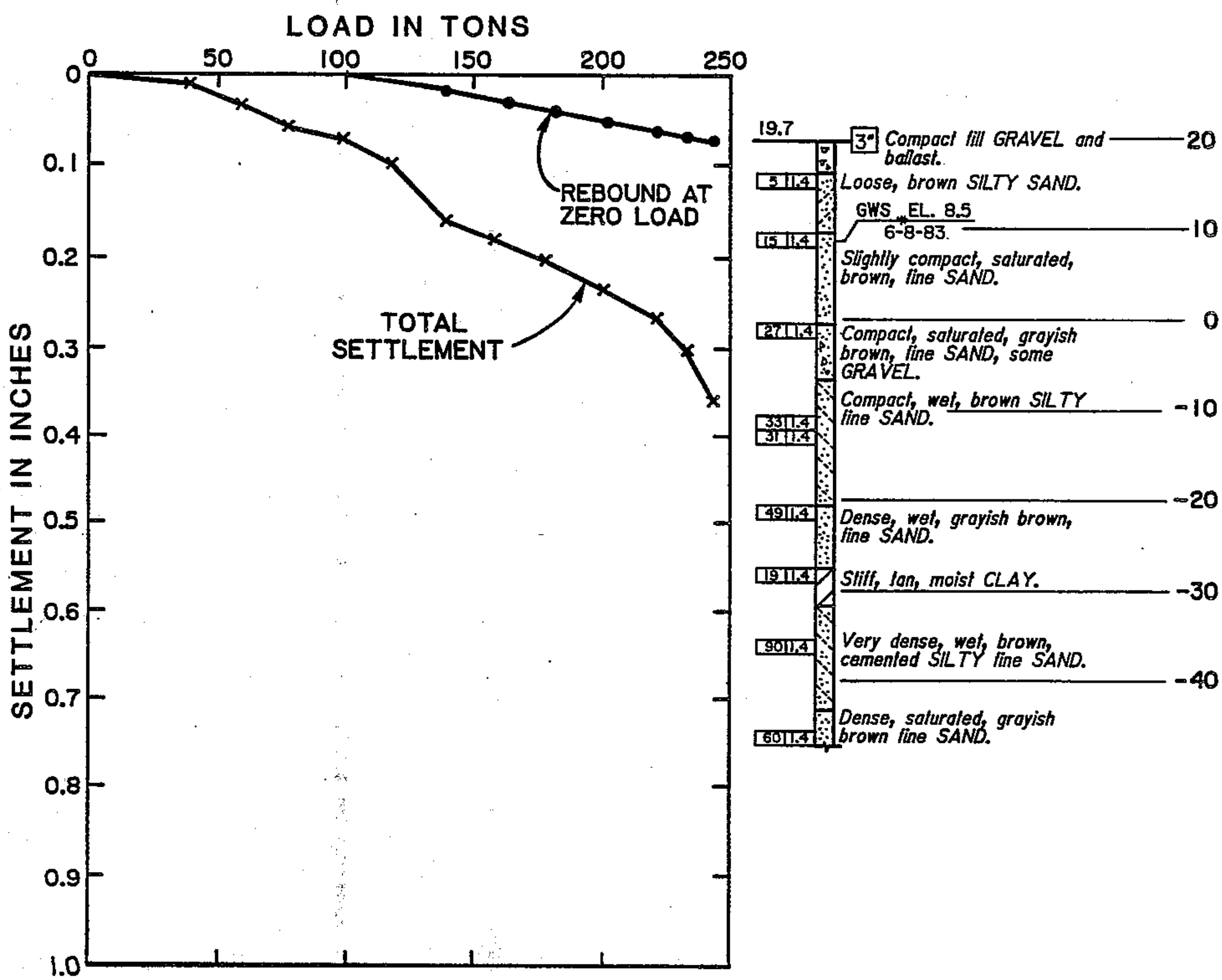
Initial Dynamic Test	175 Tons	J_c = 0.25
Restrike Dynamic Test	234 Tons (RMAX)	J_c = 0.25
Initial ENR 40 b/ft	124 Tons	
Restrike ENR (72 b/ft)	N/A	
Static Load Test	240 Tons	0.36" Settlement
Anchor Pile Pullout	120 Tons	0.53" Uplift

Restrike Period - 18 hours

Static Load Period - 3 days after restrike

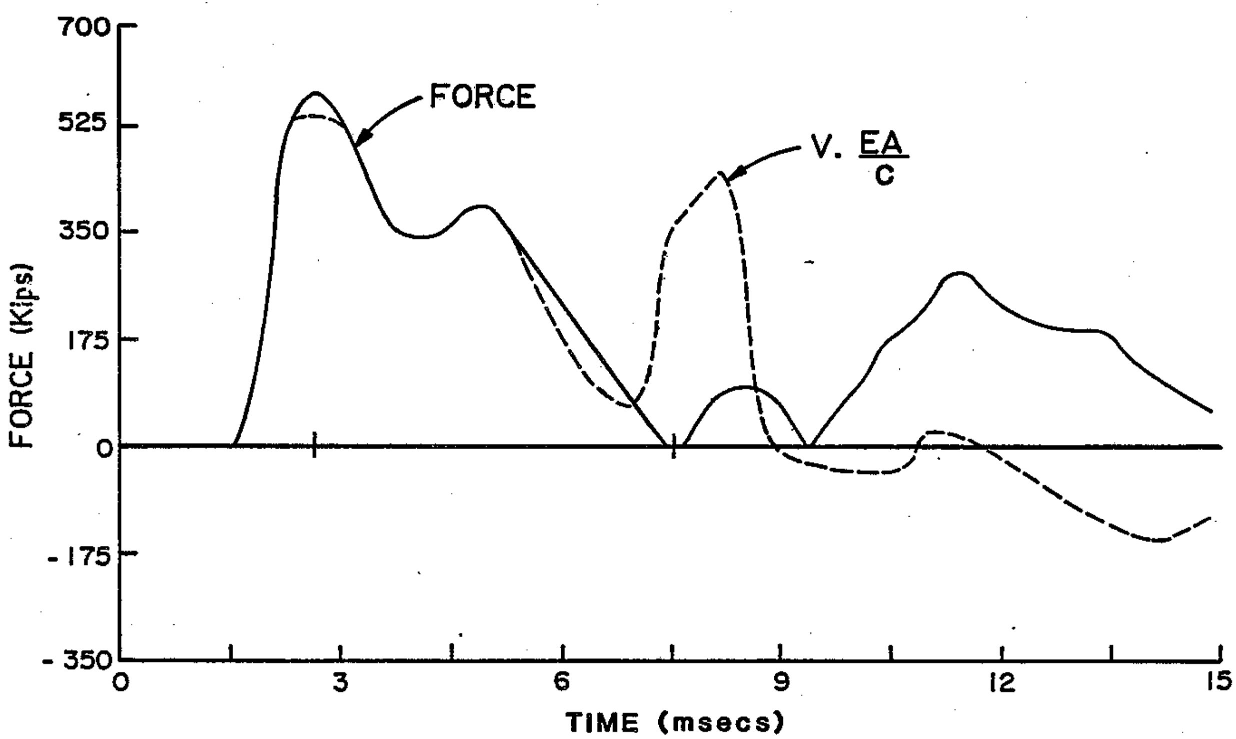
Penetration L - 26.75 ft

BEARING PILE TEST DATA
Bent 4, Sacramento Light Rail Transit, SACRAMENTO

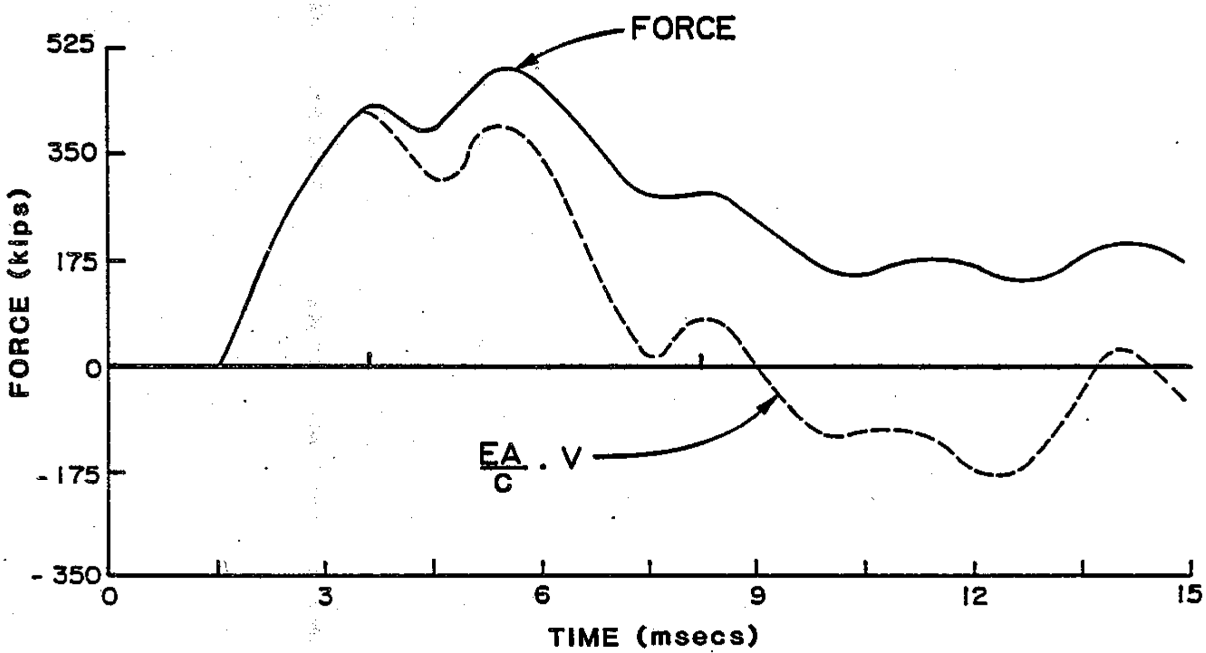


STATIC LOAD TEST RESULTS AND LOG OF TEST BORING

Bent 4, Sacramento Light Rail Transit, SACRAMENTO



INITIAL WAVE TRACES
Bent 4, Sacramento Light Rail Transit, SACRAMENTO



RESTRIKE WAVE TRACES

Bent 4, Sacramento Light Rail Transit, SACRAMENTO

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